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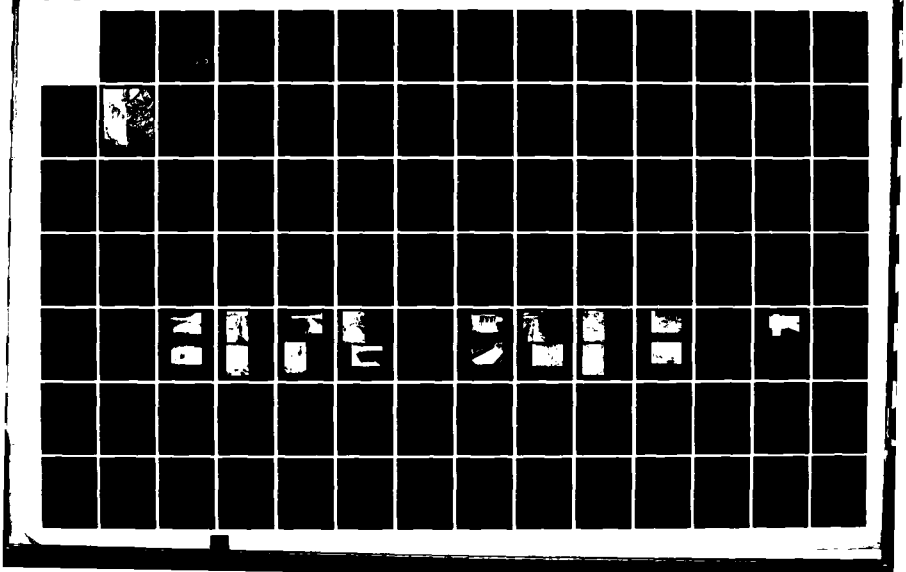

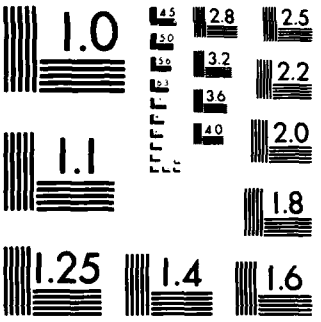

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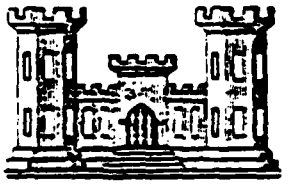
MASSACHUSETTS COASTAL AREA

WALTHAM, MASSACHUSETTS

CAMBRIDGE RESERVOIR DAM
MA 00750

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM



DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS

WALTHAM, MASS. 02154

JANUARY 1980

UNCLASSIFIED

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

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Massachusetts Coastal Area
Waltham, Massachusetts
Hobbs Brook

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is a 32.5 ft. high, 1850 ft. long earth embankment structure containing a gated masonry intake structure, an 18 ft. long masonry spillway and an indicated concrete corewall. The dam is considered to be in fair condition. The size is intermediate with a hazard potential of high. It is recommended that the owner emgage a qualified engineer to investigate seismic stability, safety of the dam with respect to the presence of water main(s).

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REPLY TO
ATTENTION OF:
NEDED

APR [illegible] 1990

Honorable Edward J. King
Governor of the Commonwealth of
Massachusetts
State House
Boston, Massachusetts 02133

Dear Governor King:

Inclosed is a copy of the Cambridge Reservoir Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Quality Engineering, the cooperating agency for the Commonwealth of Massachusetts. In addition, a copy of the report has also been furnished the owner, Cambridge Water Department.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Quality Engineering for your cooperation in carrying out this program.

Sincerely,

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

NATIONAL DAM INSPECTION PROGRAM
PHASE I INVESTIGATION REPORT
BRIEF ASSESSMENT

Identification No.: MA 00750

Name of Dam: Cambridge Reservoir Dam

City: Waltham

County and State: Middlesex County, Massachusetts

Stream: Hobbs Brook

Date of Inspection: October 30, 1979

The dam is a 32.5 foot high, 1,850 foot long earth embankment structure containing a gated masonry intake structure, an 18 foot long masonry spillway and an indicated concrete corewall. The dam was completed in 1397. The dam has always been owned and operated by the City of Cambridge as part of their water supply system.

The visual inspection generally indicated the dam to be in fair condition. Riprap on the upper part of the upstream slope was displaced in several locations. Sloughing of the slope near the crest and erosion of the spillway discharge channel were also observed. Water mains were observed along the crest and large trees were present on the downstream slope.

Since there was no indepth engineering data available, the adequacy of the dam was primarily evaluated by visual inspection, past performance history and sound engineering judgement.

The dam has a size classification of intermediate and a hazard classification of high. Based upon Corps Guidelines, the test flood would be the full PMF, which would produce an inflow

of 11,935 cfs. Considering the reservoir to be initially at its normal operational pool elevation of 181, the resulting outflow of 2,400 cfs would overtop the dam by about 0.5 feet (elevation 186.5). The combined capacity of the intake structure and spillway under these conditions would be 1,120 cfs or 47 percent of the test flood outflow.

The dam is in generally fair condition. It is recommended that the Owner engage a qualified, registered professional engineer to investigate the following:

1. Seismic Stability
2. Safety of the dam with respect to the presence of water main(s).
3. Prevention of erosion at the downstream slope from catch basin discharge.
4. Potential of overtopping and the adequacy of the spillway.
5. Removal of rubble fill, trees and brush from the downstream slope and regrading of this slope.

Furthermore, the Owner should institute remedial measures including the proposed renovations of the spillway discharge channel; the proposed repair of the riprap on the upstream slope; establishment of a system for locking stoplogs in place; testing of the gates and establishment of a formal downstream warning system.

The above recommendations and remedial measures should be instituted by the Owner within one year of receipt of this Phase I Inspection Report.

Ronald H. Cheney, P.E.
Vice President

Hayden, Harding & Buchanan, Inc.
Boston, Massachusetts

This Phase I Inspection Report on Cambridge Reservoir Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

RICHARD DIBUONO, CHAIRMAN
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation: however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to

assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

CAMBRIDGE RESERVOIR DAM

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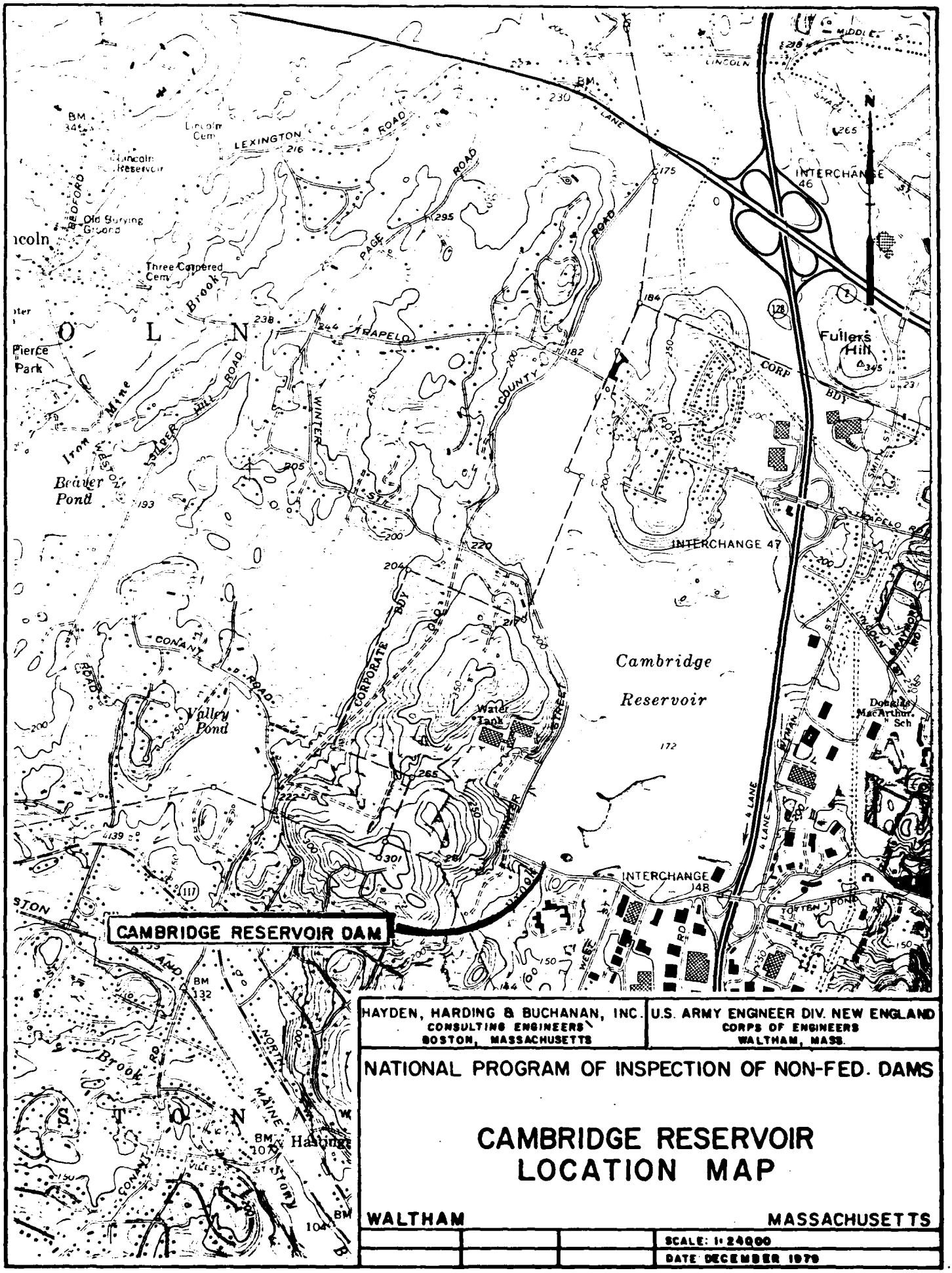


HAYDEN, HARDING & BUCHANAN, INC. CONSULTING ENGINEERS BOSTON, MASSACHUSETTS	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

CAMBRIDGE RESERVOIR LOCATION MAP

WALTHAM MASSACHUSETTS

SCALE: 1:24000

DATE DECEMBER 1979

PHASE I
NATIONAL DAM INSPECTION PROGRAM

SECTION 1
PROJECT INFORMATION

1.1 General

a. Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Hayden, Harding & Buchanan, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Massachusetts. Authorization and notice to proceed was issued Hayden, Harding & Buchanan, Inc. under a letter of 24 October 1979 from William E. Hodgson Jr., Colonel, Corps of Engineers. Contract No. DACW 33-80-C-0006 has been assigned by the Corps of Engineers for this work.

b. Purpose

(1) Perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-Federal interests.

(2) Encourage and assist the States to initiate quickly effective dam safety programs for non-Federal dams.

(3) To update, verify and complete the National Inventory of Dams.

1.2 Description of Project

a. Location

Cambridge Reservoir Dam is located in the City of Waltham in Middlesex County, Massachusetts. The resevoir is located to the northwest of the Winter Street, Route 128 intersection. The dam impounds the waters of Hobbs Brook, and is shown on the Concord, Massachusetts Quadrangle with the approximate coordinates of North $42^{o}23'51"$, West $71^{o}16'25"$.

b. Description of Dam & Appurtenances

Cambridge Reservoir Dam is a 32.5 foot high, 1850± foot long earth embankment structure containing a masonry intake structure, a masonry spillway and an indicated concrete corewall. A plan dated 1895 indicates two cross sections referred to as "Winter Street Embankment" and "Dam Section". The Dam Section (see Appendix B) has a 40 to 60 foot wide crest, a 1½ Hor.:1 Vert. downstream slope and a stepped upstream slope. At the intake structure, the upstream slope has a 17 foot high riprapped upper section on a 1½ Hor.:1 Vert. slope, a 5± foot berm and a 15.5 foot high lower section sloped on a 2 Hor.:1 Vert. slope. The typical "Winter Street Embankment" section has a 1½ Hor.:1 Vert. riprapped slope with no berms. The intake structure is shown in photograph 9 (see Appendix C). It contains an ungated arch spillway on each side and reportedly contains 3 steel gated intake openings on the upstream side. The location, size and invert elevations

of these openings are unknown. The intake structure outlets through a 72 inch inside diameter concrete culvert, photograph 7. The invert elevation of the pipe is 153.3. The intake structure has a steel frame wood deck service bridge leading from the crest.

The dimensions, location and horizontal extent of the corewall is unknown. Information obtained from plans (see Appendix B) dated 1895 indicates the top of the corewall to be at elevation 185. These plans indicate the wall to extend the length of the "Dam Section" and not within the "Winter Street Embankment". However, no differentiation between the location of the "Dam Section" and "Winter Street Embankment" was described on the plans.

Water flowing into the 18± foot long masonry spillway (photographs 10 and 11) is controlled by the 2'-2" opening between the spillway floor and the bottom of the I Beam for the roadway bridge spanning the spillway. The spillway weir is located approximately 4 feet upstream of the bridge. The weir contains provisions for 4.3 feet of stoplogs. The abutments for the weir section have a brick cap. The spillway outlet is a 36± inch diameter concrete pipe. The outlet pipe is shown by photograph 12.

c. Size Classification

The dam has a size classification of intermediate based on its storage capacity of 10,600 acre-feet.

d. Hazard Classification

The hazard potential due to dam failure flooding is classified as high. According to Corps guidelines the outflow from dam failure is 44,200 cfs. The impact area around North Avenue contains

substantial residential development. The flood stage will reach 10 to 20 feet. Seventeen homes, several roads, and two industrial buildings are within the impact area.

e. Ownership

The dam has always been owned by the Cambridge Water Department.

f. Operator

The dam is maintained by the Cambridge Water Department. Mr. John Beekmen is the designated caretaker of the dam. The address is 250 Fresh Pond Parkway, Cambridge, Massachusetts 02138. (Telephone 617-498-9070)

g. Purpose of Dam

The purpose of this dam has always been for water supply.

h. Design and Construction History

Design of the dam was completed in 1895. The dam was constructed during the years of 1895 through 1897. During 1963, the downstream slope, the spillway and the intake structure discharge outlets were modified to allow a utility line to traverse the crest of the dam. There is proposed work to decrease the steepness of the downstream slope, and improve the general condition of the dam. Camp, Dresser & McKee of Boston, Massachusetts is the engineering firm for these improvements.

i. Normal Operational Procedures

The caretaker monitors the gates to attempt to maintain the elevation of the reservoir at 180 to 181. Water discharges into Hobbs Brook, to Stony Brook Reservoir and eventually into Fresh Pond Reservoir, where it is treated and distributed into the Cambridge water system.

1.3 Pertinent Data

a. Drainage Area

The drainage area, 6.82 s.m. (4,365 acres) has a generally rolling to slightly hilly topography. The major drainage path is along Hobbs Brook, which feeds the reservoir from a large swampy area to the north. The reservoir is divided into three sections by roadway crossings. Water from the reservoir outlets into Hobbs Brook and eventually flows into Stony Brook about 1.7 miles downstream of the dam. See hydraulic calculations in Appendix D.

The area around Cambridge Reservoir is moderately to heavily developed, with a number of industrial and residential structures. State Routes 2 and 128 and a number of major roads pass through the drainage area.

Two large industrial buildings are located adjacent to Hobbs Brook between 500 and 1000 feet downstream of the dam. There is little development for the next mile downstream as the Brook flows through a park and undeveloped land. Below this point moderate residential development occurs near the Brook, extending to its confluence with Stony Brook. North Avenue crosses Hobbs Brook about 1.4 miles below the dam. See drainage area map in Appendix D, and photographs in Appendix C.

b. Discharge at Damsite

1. Outlet Works

The outlet works for this project consist of an intake structure and a spillway structure. The intake structure or overflow chamber contains two arch spillways (one on each side), see photograph 9 in Appendix C. Discharge is reportedly controlled

by 3 steel gated sluice openings on the upstream side of the overflow chamber. The locations, sizes and inverts of these gates is not known. The outflow is carried through the dam by a 72 inch reinforced concrete pipe which discharges into Hobbs Brook. The downstream invert for this pipe is at elevation 153.3. With the reservoir at its full pool elevation of 181, the discharge capacity for the outlet pipe would be approximately 970 cfs.

2. Gated Spillway Capacity

The spillway consists of a spillway weir, a rectangular bridge opening on the upstream face, and a 36 inch reinforced concrete pipe which discharges on the downstream side of the dam. The spillway weir has an ungated invert elevation of approximately 180.7 and provisions for 4.3 feet of stoplogs. The abutments and sidewalls for this structure extend about four feet from the bridge face to the weir. The roadway bridge spanning the spillway has an 18 foot long by 2.2 foot high opening between the spillway floor and bottom of the bridge. The 36 inch outlet pipe has a downstream invert of approximately 180.5 and discharges into Hobbs Brook. Under normal conditions, with a full pool elevation at 181, about 2 feet of stoplogs would be in place to prevent discharge through the spillway structure.

3. Maximum Known Flood at Damsite

No records of maximum impoundment or outlet discharges are available for this project. However, the reservoir reportedly has been operated with pool elevations of up to 183.25. Presently the reservoir pool is normally maintained at elevation 180 to 181.

There are no records indicating that the dam has ever been overtopped. United States Weather Bureau records indicate that from August 17 to 20, 1955 nine to eleven inches of rainfall occurred near the general location of the project.

4. Project Discharge at Top of Dam

For a reservoir pool elevation of 186, top of dam and roadway, the 72 inch outlet pipe from the overflow chamber could have a maximum discharge capacity of 1050 cfs and the 36 inch pipe from the spillway would have a capacity of approximately 55 cfs.

5. Total Project Discharge at Test Flood Elevation

Assuming a water level at elevation 181, the PMF inflow of 11,935 cfs would surcharge the reservoir to an elevation of 186.5. This would result in the dam and roadway being overtopped by about 0.5 feet of water. The total PMF outflow, including that through the outlet pipes and over the top of the dam, would be 2,400 cfs. The combined outflow of the overflow chamber and spillway with up to three feet of stoplogs in place would be approximately 1,120 cfs or about 47% of the total PMF outflow under these conditions. It is assumed that the outflow through these structures is controlled by the size of the outlet pipes.

c. Elevation (ft. above NGVD - approximate based on USGS map)

(1) Streambed at toe of dam ---------- 153.3
pipe outlet from overflow chamber

(2) Bottom of cutoff ---------- Unknown if any

(3) Maximum tailwater ---------- 160.0±
(for test flood outflow)

(4) Recreation pool ---------- N/A

(5) Full flood control pool ---------- N/A

(6) Spillway crest ---------- 180.7

(7) Design surcharge (Original Design) ---------- 181.0±

(8) Top of dam ---------- 186.0

(9) Test flood surcharge ---------- 186.5

d. Reservoir (Length in feet)

(1) Normal pool ---------- 15,000±

(2) Top of dam ---------- 17,500±

(3) Test flood pool ---------- 17,700±

(4) Flood control pool ---------- N/A

(5) Spillway crest pool ---------- N/A

e. Storage (acre-feet)

(1) Normal pool ---------- 10,600
(water supply)

(2) Spillway crest pool ---------- 10,600

(3) Top of dam ---------- 15,400

(4) Test flood pool ---------- 15,800

(5) Flood control pool ---------- N/A

f. Reservoir Surface (acres)

(1) Normal pool ---------- 948±

(2) Spillway crest ---------- 948±

(3) Top of dam ---------- 1093±

(4) Test flood pool ---------- 1111±

(5) Flood-control pool ---------- N/A

g. Dam

(1) Type ---------- gravity, earth embankment

(2) Length ---------- 1850±'

(3) Height ---------- 32.5±'

(4) Top Width ---------- varies 40 to 60± feet

(5) Side Slopes -- U.S. 1½ Hor.:1 Vert. to 2 Hor.:1 Vert.
D.S. 1½ Hor.:1 Vert.±

(6) Zoning ------- indicated on original design plans (dated 1895); location & extent are unknown

(7) Impervious Core ------ concrete corewall indicated on original design plans (dated 1895); location, dimensions & extent are unknown

(8) Cutoff ---------- none indicated

(9) Grout curtain ---------- none indicated

h. Diversion and Regulating Tunnel ---- none at this project

i. Spillway

(1) Type ---------- masonry, broad crested

(2) Length of weir ---------- 18'

(3) Crest elevation (with and without stoplogs) --------
180.7± feet without stoplogs
182.6± feet with stoplogs

(4) Gates ---------- N/A

(5) U/S Channel ---------- None

(6) D/S Channel ---------- unlined channel badly eroded

(7) General ---------- downstream outlet is 36" RCP

j. Regulating Outlets

The regulating outlets for this dam are the overflow chamber and spillway. The intake structure or overflow chamber reportedly contains 3 steel gates which are manually operated by valves within the structure. No data was located to indicate the size, location and invert of these gated openings. The chamber outlets through a reinforced concrete pipe. The pipe has a 72 inch inside diameter and invert elevation of 153.3 at the downstream face of the dam. The overflow chamber contains 2 ungated arch spillways, one on each side. The invert of these spillways is unknown.

The spillway consists of a masonry broad crested weir with provisions for stoplogs, a rectangular opening between the spillway crest and bottom of a roadway bridge and a 36" reinforced concrete outlet pipe. There are two masonry abutment walls which extend about 4 feet (towards the reservoir) from the bridge opening to the weir location. There are provisions for the manual placement of up to 4.3 feet of stoplogs at the weir. The bridge opening has dimensions of 18 feet by 2.2 feet with an invert elevation of about 180.7. The 36" RC pipe outlets on the downstream side of the dam and has an invert elevation of 180.5.

SECTION 2

ENGINEERING DATA

2.1 Design Data

A limited number of plans, dated 1895, were located at the office of the Cambridge Department of Public Works. No indepth design calculations were located and no information was found indicating by whom the dam was designed. Modifications were made to the downstream embankment and outlet structures of the dam and dike for the installation of a utility line in 1963. No plans or design calculations are available for this work. Maintenance work on the roadway atop the dam (Winter Street) and modifications to the downstream embankment, downstream outlets and outlet channels are proposed to be undertaken in late 1979. Design plans, dated August, 1979, were obtained from Camp, Dresser & McKee, Boston, Massachusetts, the engineering consultants for this work.

2.2 Construction Data

The reservoir was built between 1895 and 1897. No construction data was located. In 1963, additional fill was placed on the downstream embankment of the dam to facilitate the installation of utility lines. Modifications of the existing structure including placement of additional fill on the downstream embankment, modifications and extensions to the outlet conduits for the overflow chamber and spillway, and the installation of riprapped channels downstream of these outlets are proposed to be undertaken in late 1979 as stated in section 2.1 above.

2.3 Operation Data

The structure is operated by a designated caretaker employed by the Cambridge Water Department. The caretaker regulates outflow through gates within the overflow chamber to maintain a desired reservoir elevation of 180 to 181. There is no written formal operational manual for this structure.

2.4 Evaluation of Data

a. Availability

A limited number of plans were available at the office of the Cambridge Department of Public Works. The design plan for the proposed modifications and maintenance work scheduled for late 1979 was provided by the Boston office of the engineering consulting firm of Camp, Dresser & McKee, Inc. No County or State Inspection Reports were available for this dam.

b. Adequacy

The lack of *indepth engineering* data does not allow for a definitive review. Therefore, the adequacy of this dam, structurally and hydraulically, can not be assessed from the standpoint of review of design calculations, but must be based primarily on the visual inspection, past performance history, and sound engineering judgement.

c. Validity

The field investigation indicated that the external features of the embankment dam substantially agree with those shown on available plans. Due to the modifications to the downstream embankment and outlet structures in 1963, the existing plans do not exactly agree with these features of the dam as they exist today. The outlet pipe from the intake structure was measured in the field to have a 72 inch inside diameter. The plans prepared by Camp, Dresser & McKee, Inc. indicate the pipe to have a 84 inch diameter.

SECTION 3

VISUAL INSPECTION

3.1 Findings

a. General

At the time of inspection the water in the reservoir was about 4.5 ft. below the top of the dam.

b. Dam

The dam consists of an earth embankment about 1,850 ft. long and about 32.5 ft high with an intake structure and a spillway structure. The foundation material of the dam is unknown.

Upstream Slope

The upper 4 to 5 ft. of the upstream slope was above the reservoir level and available for inspection. A general view of the entire upstream slope is shown in photograph 8. Two types of riprap slope protection were observed, as shown in photograph 3. The upper 2.5 ft. ± consists of a nearly vertical wall of hand placed angular boulders and cobbles; below this are hand placed cut stone pieces about 8 in. thick and ranging in size from about 1.5 ft. by 1.5 ft. to about 4 ft. by 4 ft. The cut stone pieces are sloped about 1.5 Hor.:1 Vert.

The upper riprap wall is displaced in several locations, the most deterioration being from the intake structure bridge to about 110 ft. right of the bridge, photograph 3. Photographs 2 and 4 shows undermining of the crest about 50 ft. and 90 ft. right of the bridge, respectively.

The cut stone riprap is in good condition; only minor displacements were observed.

Crest

The crest, covered with an asphalt pavement, has an average width of about 50 ft., photograph 1. The pavement is in generally good condition but does have several longitudinal cracks near the centerline.

No significant misalignment of the guardrail on the upstream side of the crest was observed. A small amount of brush growth and several tree stumps up to 2 ft. in diameter were observed on the upstream side of the pavement.

The downstream edge of the crest is irregular, partly as a result of dumping onto the downstream slope. Fire hydrants, catch basins, and wood utility poles were observed along the downstream side of the pavement.

Downstream Slope

Generally, the downstream slope is uneven and is in poor condition. Dumping has occurred on the slope resulting in a cover of undesirable rubble such as tree trunks and limbs, concrete, asphalt, and scrap metal, photograph 13. Heavy brush and tree growth was evident on the downstream slope, photograph 14.

Discharge pipes from catch basins were observed at the top of the slope, photograph 13.

An area of standing water downstream of the toe, photograph 15, was attributed to storm water runoff. No evidence of seepage through the dam was observed.

c. Appurtenant Structures

The intake structure, shown in photographs 8 & 9, routes water to a 72" diameter reinforced concrete pipe, which passes through the dam and outlets into Hobbs Brook. The service bridge

to the intake structure has a steel I beam frame with a wood deck and steel handrail. All components were observed in generally good condition. Boulders and cobbles were observed at the downstream end of the outlet pipe, as shown in photograph 7. This photograph also shows dumped rock on the downstream slope to the right of the outlet pipe.

The spillway, about 150 ft. left of the intake structure, routes water to a 36 in. diameter pipe which passes through the dam. The spillway discharge channel, photograph 16 is approximately parallel with the dam until it meets the outlet works discharge channel. The banks of the spillway discharge channel are unprotected and erosion of the sides has occurred.

d. Reservoir Area

There are no indications of instability along the banks of the reservoir in the vicinity of the dam.

e. Downstream Channel

The downstream channel is the natural riverbed, photograph 6. No significant obstructions existed in the channel at the time of inspection.

3.2 Evaluation

Visual inspection indicates the dam to be in generally fair condition.

Riprap on the upper part of the upstream slope has been displaced in several locations, and sloughing of the slope near the crest has occurred in some of these locations. The downstream slope is partially covered with dumped rubble. The banks of the spillway discharge channel are eroding which, if allowed to continue, could cause instability of the dam.

SECTION 4

OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 Operational Procedures

a. General

The Cambridge Reservoir Dam is owned by the Cambridge Water Department. The designated caretaker is Mr. John Beekmen. As the purpose of the reservoir is for water supply, the caretaker regulates the flow through the intake gates in the overflow chamber in order to maintain a desired full pool elevation of 180 to 181. Outflow can also be regulated at the spillway structure which has provisions for up to 4.3 feet of stoplogs.

b. Description of Warning Systems

There are no warning systems at this dam.

4.2 Maintenance Procedures

a. General

The Cambridge Water Department is responsible for the maintenance of this dam. At the present time, maintenance work is proposed on the downstream embankment of the dam, the outlet structures, and the roadway upon the dam crest. This work will consist of the placing of additional fill to improve the slope of the downstream embankment, maintain and improve the downstream outlets and channel, and replace or repair the existing guard rails. Additional proposed future work will include the repair and extension of existing riprap on the upstream face of the dam.

b. Operating Facilities

There is no formal maintenance procedure for this facility. The dam is used for water supply on a daily basis. Most deficiencies in the operational facilities could be detected during normal operating procedures.

4.3 Evaluation

Although there are no formal written operational or maintenance procedures, the Water Department periodically removes debris from the spillway and performs general maintenance. The structure should be inspected every year by a registered professional engineer who can identify conditions of concern which, if left unchecked, could jeopardize the safety of the structure.

SECTION 5

EVALUATION OF HYDRAULIC/HYDROLOGIC FEATURES

5.1 General

Cambridge Reservoir Dam is located in the City of Waltham, Massachusetts, and the impounded reservoir extends from Waltham into the adjacent Towns of Lincoln and Lexington. The facility is used to impound water from Hobbs Brook for water supply purposes. At the normal pool elevation of 181, it has a storage capacity of 10,600 acre-feet and surface area of 948 acres.

The reservoir has a drainage area of 6.82 square miles (4,365 acres), comprised of rolling hills and several swampy areas. The largest of these swamps (280± acres) is located about 3 miles to the north of the dam, and is the source of Hobbs Brook. This swampy area could significantly affect the rate of storm runoff to the reservoir.

The drainage area is intercepted by 2 major roadways (Trapelo Road and Route 2). The embankments at these roadways contain culverts which equalize the water level on each side. There is an old gatehouse structure located at the Trapelo Road crossing which is no longer operational. See photograph 17 in Appendix C.

Water can be discharged through an overflow chamber located at the southern end of the reservoir into Hobbs Brook. This brook flows southerly for about 1.4 miles to its confluence with Stony Brook, which flows southeasterly until it joins the Charles River. A map of the drainage area along with plans and sketches of the structure and its outlets is contained in Appendixes B and D.

Additional information on the drainage area and reservoir can be found in Sections 1.2 and 1.3. Photographs of the facility are shown in Appendix C.

5.2 Design Data

The original facility was completed in 1897. Design calculations were not located but a limited number of plans showing the proposed 1897 work were found. Plans showing proposed modifications and maintenance work to be undertaken in late 1979 and 1980 were obtained from the consultants for this project, Camp, Dresser & McKee of Boston, Massachusetts. The reservoir was designed and has always been used for water supply.

5.3 Experience Data

Records of past flood experiences could not be found. Reportedly the dam has never been overtopped. During the period of August 17 to 20, 1955, records from the U.S. Weather Bureau indicate that between 9 and 11 inches of rainfall occurred in the general vicinity of the Cambridge Reservoir.

5.4 Test Flood Analysis

The dam has an intermediate size classification and a high hazard potential. Based upon Corps Guidelines, the test flood would be the PMF. The test flood inflow was determined to be 11,935 cfs. This considers runoff from the 6.82 s.m. "rolling" drainage area to be 1,750 cfs. Roadway crossings were considered to not significantly influence runoff patterns of the PMF. See photograph 17 in Appendix C.

Outflow from the reservoir is regulated by the gates in the overflow chamber connected to a 72 inch outlet pipe and the spillway, connected to a 36 inch outlet pipe. No information is available as to the size, type, and locations of the gates, so the 72 inch outlet pipe was used to determine the discharge capacity of the overflow chamber. Normally, up to 2 feet of stoplogs are in place at the spillway. At the full reservoir elevation of 181, the outflow through the 72 inch pipe would be about 970 cfs while stoplogs would prevent discharge across the spillway. Photographs 9 to 12, and 7 in Appendix C show these structures. Hydraulic calculations are contained in Appendix D.

With the initial water level at elevation 181±, the test flood inflow of 11,935 cfs would surcharge the reservoir to elevation 186.5±. The resulting outflow would be approximately 2,400 cfs. The overflow chamber and spillway would have a combined capacity of 1,120± cfs or 47% of the outflow. The remaining flow would overtop the dam by about 0.5 feet. The reservoir would provide stage storage for approximately 15.2 inches or 5,200 acre-feet of runoff.

5.5 Dam Failure Analysis

The failure analysis was performed assuming an initial reservoir level at elevation 186, top of dam. The dam has a hydraulic height of 32.5 feet and a maximum storage capacity of 15,400 acre-feet. Immediately before dam failure, the overflow chamber and spillway would be releasing a combined discharge of approximately 1,100 cfs. This flow could flood up to 6 houses near North Avenue by 1 to 5 feet of water, but would not overtop that roadway.

Using Corps "rule of thumb" guidance, the failure of the dam would result in a peak outflow of 44,200 cfs. Six industrial buildings located between 500 and 1,000 feet downstream of the dam would be inundated by 10 to 16 feet of water. Between these industrial buildings and North Avenue at least 6 additional houses would be damaged to varying extents by floodwater depths between 2 and 5 feet. Approximately 7,700 feet below the dam, North Avenue with an earthen road embankment crosses the outlet brook. A rectangular concrete culvert with dimensions of 7' by 9' passes through the embankment. The top of the embankment is at elevation 113±. This constriction of the flood plain could cause a backwater condition upstream of North Avenue. The structural integrity of the embankment may be seriously reduced by a high water level on its upstream face. At North Avenue, the six homes damaged by base flow flooding would receive additional failure flood damage. Dam failure flood stage could reach depths of about 20 feet. The North Avenue embankment would be overtopped by up to 6 feet of water, and could possibly fail as a result. Another 9 houses, portions of several improved roads, and a rail line would be inundated by 5 to 10 feet of floodwater in the area beyond North Avenue. Loss of life and substantial property damages could occur as a result of the failure of this dam.

SECTION 6

EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observation

The visual observations did not disclose any immediate stability problems. However, several problems were observed which, if allowed to continue, could lead to instability of the dam in the future. These are:

a. deterioration of the upper 2 to 3 ft. of riprap on the upstream slope.

b. dumping of rubble on the downstream slope.

c. catch basins on the crest with discharge pipes to the downstream slope; concentrated flow of water over the unprotected surface of the downstream slope could cause erosion of the dam.

d. the presence of water mains in the dam; a water main leak could cause erosion of the dam.

e. large trees on the downstream slope.

f. erosion of the banks of the spillway discharge channel.

6.2 Design and Construction Data

Plans dated 1895 indicate an embankment cross section consisting of a 1.5 Hor.:1 Vert. upstream slope, 40 ft. wide crest at EL. 186, and a 2 Hor.:1 Vert. downstream slope. The upstream part of the dam is noted to consist of "selected blue gravel" and the downstream part of "gravel".

The 1895 plans indicate a concrete corewall to EL. 185; however, the plans do not indicate the dimensions or location of this wall.

A drawing showing proposed improvements to the dam, dated August 1979, was made available. Proposed improvements include, but are not limited to, the following:

1. Repair of riprap on upstream slope.
2. Filling, grading, and seeding downstream slope.
3. Lining spillway discharge channel.

6.3 Post Construction Changes

The steepness of the downstream slope, the spillway outlet and the intake structure outlet were modified in 1963.

6.4 Seismic Stability

The dam is located near the boundary of Seismic Zones 2 and 3 and in accordance with the recommended Phase I guidelines warrants seismic analysis. No record of seismic analysis made by conventional equivalent static load methods were available.

SECTION 7

ASSESSMENT, RECOMMENDATIONS & REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition

The visual inspection indicates that the dam is in generally fair condition.

b. Adequacy of Information

The information made available, along with the visual inspection, is adequate for a Phase I level of investigation.

c. Urgency

The recommendations and remedial measures of Sections 7.2 and 7.3 should be implemented within one year after receipt of this Phase I Inspection Report by the Owner.

7.2 Recommendations

a. In accordance with the recommended Phase I guidelines, the dam should be analyzed for seismic stability. A qualified registered professional engineer should perform the stability analysis.

b. A qualfied registered professional engineer should analyze the safety of the dam with respect to the presence of water main(s) passing through the dam and recommend appropriate corrective measures, if necessary. Appropriate designs should be made for preventing erosion of the downstream slope from catch basin discharges.

c. The dam's spillway does not have the capacity to pass the full PMF test flood. The Owner should engage the services of a qualified registered professional engineer to further evaluate the potential for overtopping and the adequacy of the spillway.

d. A qualified registered professional engineer should supervise the removal of rubble fill, trees, and brush from the downstream slope. The slope should be regraded and grassy vegetation established. The grass should be cut as part of routine maintenance. Trespassing on the downstream slope should be prevented.

7.3 Remedial Measures

a. Operation and Maintenance Procedures

1. Proposed renovations (see Section 4.2.a.) of the spillway discharge channel should be undertaken to prevent erosion of the channel floor and banks.

2. Proposed repairs (see Section 4.2.a.) of the riprap on the upstream slope should be made.

3. A system for locking stoplogs in place should be established at the spillway stoplog structure to prevent unauthorized removal.

4. The size and location of the intake structure gates should be determined and the gates should be tested at least once a year.

5. A formal warning system should be developed for warning downstream residents in case of emergency.

6. The dam should be inspected every year by a qualified registered professional engineer who can identify conditions of concern which if left unchecked could jeopardize the safety of the dam.

7.4 Alternatives

There are no practical alternatives for this project.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST
DAMS OF NEW YORK

PROJECT CAMBRIDGE RESERVOIR DAM

DATE Oct. 30, 1979

TIME 1 pm

WEATHER 45°, Partly sunny

W.S. ELEV. 181+ U.S. ____ DN.S.

PERSONNEL

1. R. Cheney, HHB
2. D. Vine, HHB
3. D. LaGatta, GEI
4. T. Keller, GEI
5. ____
6. ____
7. ____
8. ____
9. ____
10. ____

	PROJECT FEATURE	INSPECTED BY	REMARKS
1.	Embankment	All	
2.	Intake Structure	R. Cheney, D. Vine	
3.	Spillway	R. Cheney, D. Vine	
4.			
5.			
6.			
7.			
8.			
9.			
10.			

PERIODIC INSPECTION CHECKLIST

PROJECT CAMBRIDGE RESERVOIR DAM DATE Oct. 30, 1979

FEATURE Embankment NAME D. LaGatta

DISCIPLINE Geotechnical Engineer NAME R. Cheney
Structural Engineer

AREA EVALUATED	CONDITION
DAM EMBANKMENT	
Crest Elevation	186.0
Current Pool Elevation	181±
Maximum Impoundment to Date	Unknown
Surface Cracks	None of significance observed.
Pavement Condition	Good, some longitudinal cracks near centerline.
Movement or Settlement of Crest	None observed.
Lateral Movement	None observed.
Vertical Alignment	No vertical misalignment observed.
Horizontal Alignment	No horizontal misalignment observed.
Condition of Abutment and at Concrete Structures	Good.
Indications of Movement of Structural Items on Slopes	None.
Trespassing on Slopes	Dumping miscellaneous garbage, concrete, and asphalt on downstream slope.
Sloughing or Erosion of Slopes or Abutments	Sloughing and erosion of both slopes; undermining of upstream slope in some locations.
Rock Slope Protection - Riprap Failures	Upper few feet of riprap is displaced.
Unusual Movement or Cracking at or Near Toe	Toe covered by dumped material.
Unusual Embankment or Downstream Seepage	Wet area downstream of toe attributed to storm water runoff.
Piping or Boils	None observed.
Foundation Drainage Features	None observed.
Toe Drains	None observed.
Instrumentation	None known.
Vegetation	Tree stumps on upstream side of crest, trees and brush on downstream slope.

ECHNIC [illegible]

DAM: CAMBRIDGE RESERVOIR DAM — DATE: October 30, 1979

SUBJECT FEATURE: Intake Channel & Structure — NAME: R. Cheney

DISCIPLINE: Geotechnical Engineer — NAME: D. LaGatta

Structural Engineer

AREA EVALUATED	CONDITION
OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE	
a. Approach Channel	No approach channel, intake structure is in reservoir.
Slope Conditions	
Bottom Conditions	
Rock Slides or Falls	
Log Boom	
Debris	
Condition of Concrete Lining	
Drains or Weep Holes	
b. Intake Structure	The intake structure is a stone masonry structure. The portion that could be observed appeared to be in good condition.
Condition of Concrete	
Stop Logs and Slots	

POWER CAMBRIDGE RESERVOIR DAM DATE October 30, 1979

AREA OF INSPECTION Control Tower NAME R. Cheney

DISCIPLINE Structural Engineer NAME D. Vine

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	Intake structure and control tower are one and same.
a. Concrete and Structural	
General Condition	
Condition of Joints	
Spalling	
Visible Reinforcing	
Rusting or Staining of Concrete	
Any Seepage or Efflorescence	
Joint Alignment	
Unusual Seepage or Leaks in Gate Chamber	
Cracks	
Rusting or Corrosion of Steel	
b. Mechanical and Electrical	
Air Vents	
Float wells	
Crane Hoist	
Elevator	
Hydraulic System	
Service Gates	
Emergency Gates	
Lightning Protection System	
Emergency Power System	
Wiring and Lighting System	

CAMBRIDGE RESERVOIR DAM DATE October 30, 1979

[illegible] Transition & Conduit NAME R. Cheney

DISCIPLINE Structural Engineer NAME D. Vine

[illegible]	CONDITION
[illegible] - TRANSITION AND CONDUIT	72" outlet pipe from the intake structure is underground below the dam embankment.
General Condition of Concrete	
Rust or Stains on Concrete	
Spalling	
Erosion or Cavitation	
Cracking	
Alignment of Monoliths	
Treatment of Joints	
Condition of Monoliths	

	CAMBRIDGE RESERVOIR DAM		October 30, 1979
	Outlet Structure & Channel		R. Cheney
DISCIPLINE	Structural Engineer	NAME	D. LaGatta
	Geotechnical Engineer		

OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	There is no outlet structure.
General Condition of Concrete	
Rust or Staining	
Spalling	
Erosion or Cavitation	
Visible Reinforcing	
Any Seepage or Efflorescence	
Condition at Joints	
Drain holes	None observed.
Channel	
a. Slope Protection or Lining Downstream Channel	None of significance
Condition of Discharge Channel	Fair.

VISUAL INSPECTION CHECK LIST

NAME OF DAM: CAMBRIDGE RESERVOIR DAM — DATE: October 30, 1979

PRIMARY FEATURE: Spillway — NAME: R. Cheney

DISCIPLINE: Structural Engineer — NAME: D. LaGatta

Geotechnical Engineer

AREA EVALUATED	CONDITION
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a. Approach Channel	Approach channel is the reservoir.
General Condition	
Loose Rock Overhanging Channel	None.
Trees Overhanging Channel	None.
Floor of Approach Channel	None.
b. Weir and Training Walls	
General Condition of Concrete	Brick and Stone Masonry
Rust or Staining	None Observed
Spalling	Slight deterioration of brick deck at spillway weir.
Any Visible Reinforcing	None Observed
Any Seepage or Efflorescence	Some Observed
Drain Holes	None.
c. Discharge Channel	
General Condition	Overgrown with vegetation, erosion of sides.
Loose Rock Overhanging Channel	None.
Trees Overhanging Channel	Small trees overhang channel.
Floor of Channel	Bouldery, vegetated.
Other Obstructions	None.

PROJECT	CAMBRIDGE RESERVOIR DAM	DATE	October 30, 1979
FACILITY/FEATURE	Service Bridge	NAME	R. Cheney
DISCIPLINE	Structural Engineer	NAME	D. Vine

AREA EVALUATED	CONDITION
OUTLET WORKS - SERVICE BRIDGE	
a. Super Structure	The service bridge had a steel I beam frame, wood deck and steel handrail. All components were in good condition.
Bearings	
Anchor bolts	
Bridge seat	
Additional Members	
Underside of deck	
Secondary Bracing	
Deck	
Curbs/Walks	
Railing	
Expansion Joints	
Paint	
b. Abutments/Piers	
General condition of concrete	Good
Spalling/Scaling	Good
Visible Cracking	Good
Visible Deterioration	

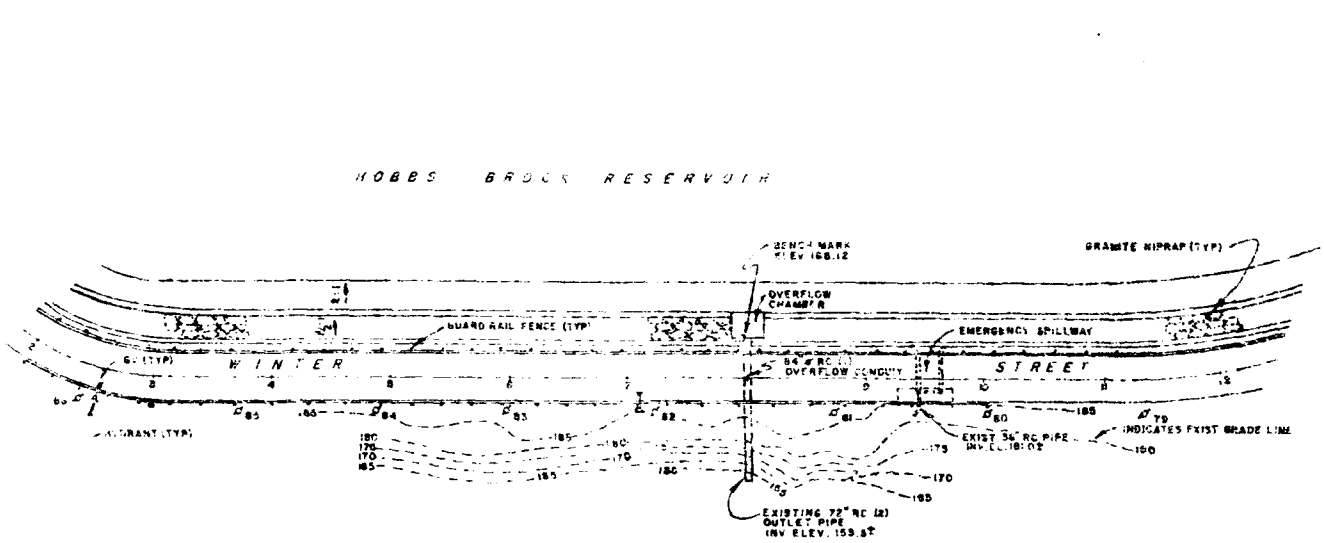
APPENDIX B

ENGINEERING DATA

LIST OF ENGINEERING DATA

1. Limited Design Plans are available at the City of Cambridge Department of Public Works, 147 Hampshire Street, Cambridge, Massachusetts.

2. Design Plans for proposed Reservoir Improvements for Hobbs Brook Reservoir are available at the office of Camp, Dresser & McKee, Consulting Engineers, 1 Center Plaza, Boston, Massachusetts.



PLAN VIEW

PLAN TAKEN FROM PLANS BY CAMP, DRESSER & McKEE, INC. DATED AUGUST 1979 (PROPOSED IMPROVEMENTS)

(1) DIAMETER OF 84" RC OUTLET PIPE TAKEN FROM COM PLANS.

(2) DIAMETER OF EXISTING 72" RC OUTLET PIPE MEASURED DURING FIELD INSPECTION.

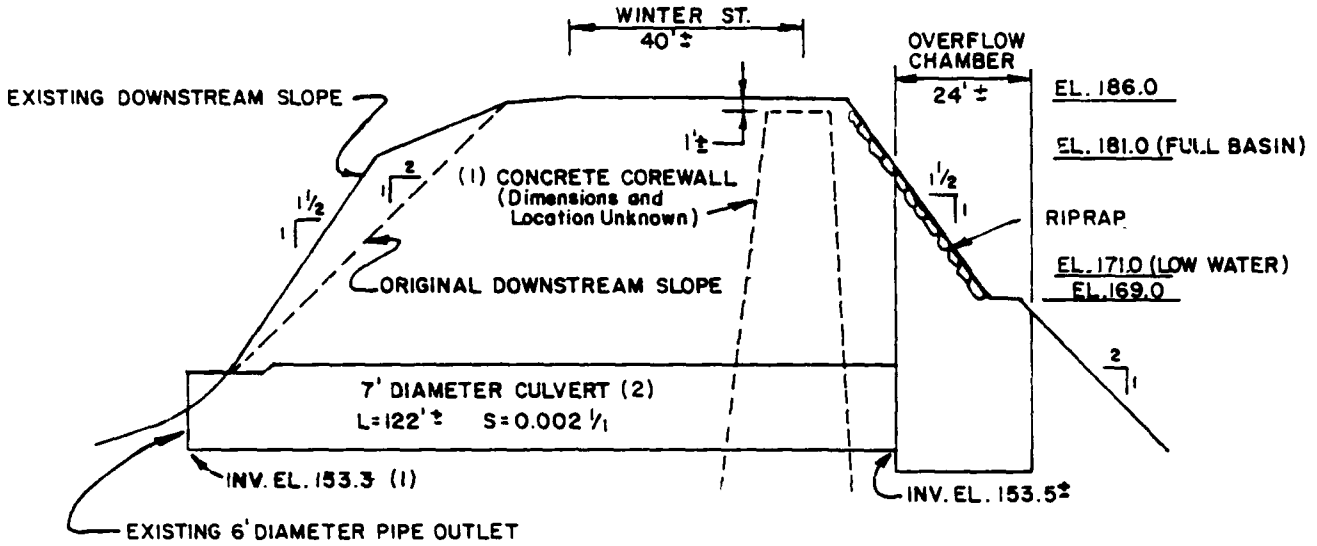
HAYDEN, HARDING & BUCHANAN, INC. CONSULTING ENGINEERS BOSTON, MASSACHUSETTS	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

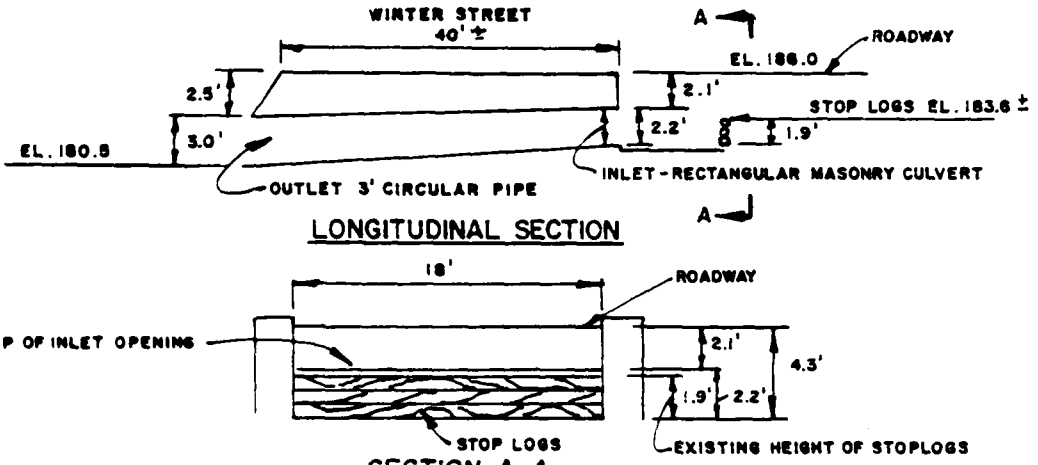
CAMBRIDGE RESERVOIR
PLAN VIEW

WALTHAM

B-3



CROSS SECTION THROUGH INTAKE STRUCTURE



SECTION A-A AUXILIARY SPILLWAY

PLAN DEVELOPED FROM EXISTING RECORDS AND ON-SITE INSPECTION

(1) FROM PLANS DATED 1895

(2) FROM PLANS BY CAMP DRESSER & McGEE DATED 1979.

HAYDEN, HARDING & BUCHANAN, INC. CONSULTING ENGINEERS BOSTON, MASSACHUSETTS	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

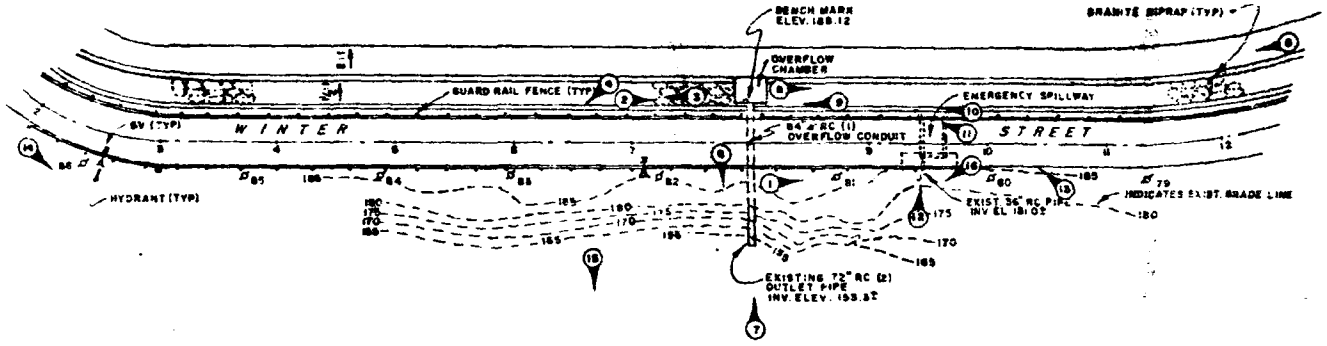
CAMBRIDGE RESERVOIR SECTION THROUGH DAM & SPILLWAY

WALTHAM MASSACHUSETTS

			SCALE: NOT TO SCALE
			DATE DECEMBER 1979

APPENDIX C

PHOTOGRAPHS



PLAN TAKEN FROM PLANS BY CAMP, DRESSER & McKEE, INC. DATED AUGUST 1979 (PROPOSED IMPROVEMENTS)

(1) DIAMETER OF 84" RC OUTLET PIPE TAKEN FROM CDM PLANS

(2) DIAMETER OF EXISTING 72" RC OUTLET PIPE MEASURED DURING FIELD INSPECTION.

HAYDEN, HARDING & BUCHANAN, INC. CONSULTING ENGINEERS BOSTON, MASSACHUSETTS	U.S. ARMY ENGINEER DIV NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

CAMBRIDGE RESERVOIR PHOTO LOCATIONS

W. LYMAN MASSACHUSETTS

C-2

PHOTO NO. 1 - Crest of Dam as viewed from intake structure looking toward left abutment.

PHOTO NO. 2 - Undermining of crest and displaced riprap on upstream slope, approximately 50 ft. right of intake structure (pencil is 6 inches long).

PHOTO NO. 3 - Displaced riprap on upstream slope looking toward right abutment. Clipboard in photo is about 110 ft. right of intake structure.

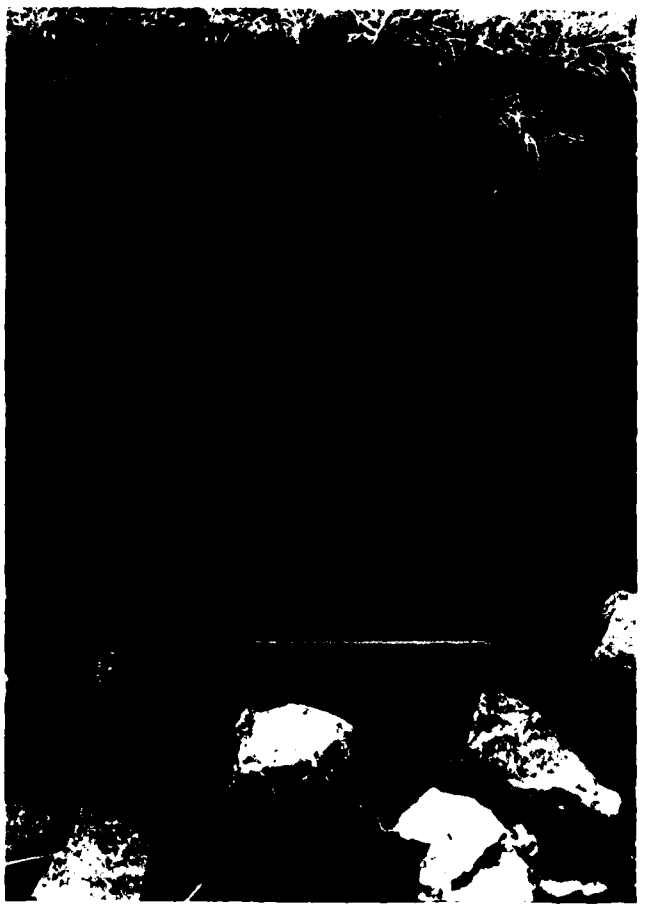


PHOTO NO. 4 - Undermining of crest above displaced riprap on upstream slope, approximately 90 ft. right of intake structure (pencil is 6 inches long).

PHOTO NO. 5 - View along crest of Main Dam. Note the location of the sidewalls for overflow spillway. The spillway is located approximately 150 ft. to the left of the intake structure.

PHOTO NO. 6 - Downstream Channel as viewed from crest.

PHOTO NO. 7 - Outlet pipe at downstream toe.

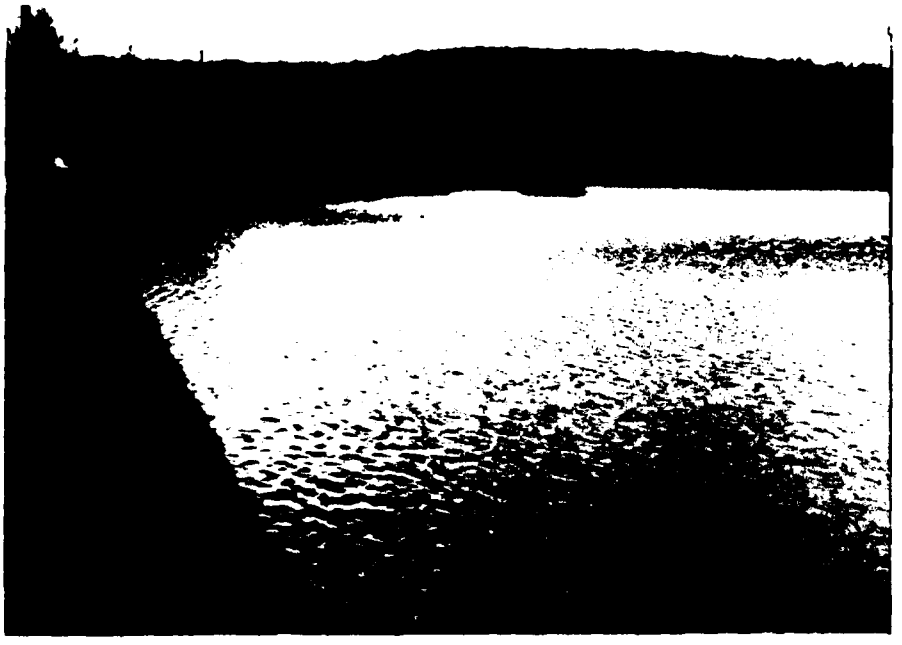


PHOTO NO. 8 - Upstream slope of Dam viewed from left abutment area.

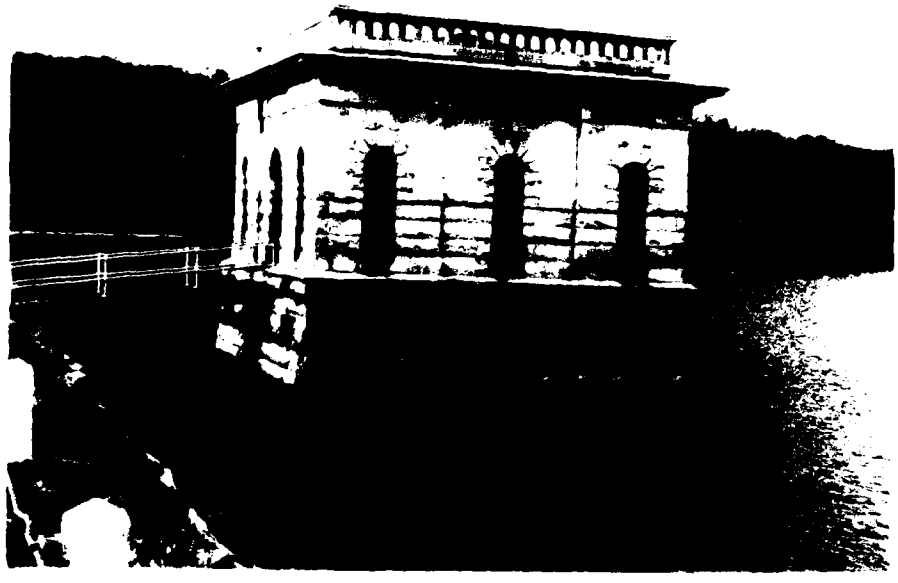


PHOTO NO. 9 - View of intake structure and service bridge. Note location of inlets to structure. Information pertaining to the location, size and inverts of the gates inside structure is not available.

PHOTO NO. 10 - Spillway floor and opening below I beam of Winter Street Roadway Bridge. A comparison of the existing structure and plans dated 1895 indicate that the spillway has undergone some modifications during the lifetime of the Dam.

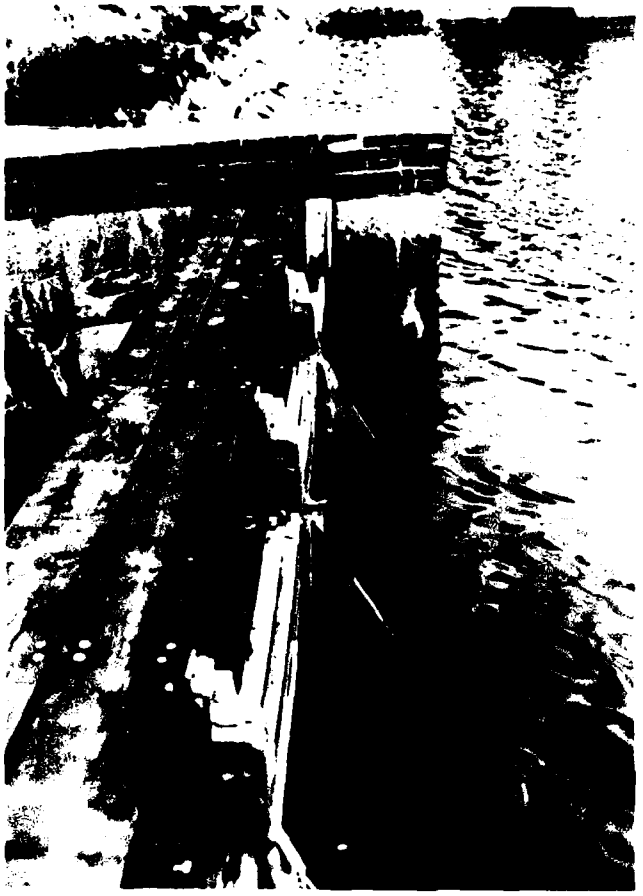


PHOTO NO. 11 - View of stop log facility for spillway. At time photo was taken there were approximately 1.9 ft. of stop logs in place. Note masonry side-wall in upper portion of picture which is also visible in Photo No. 5.

PHOTO NO. 12 - View of 3 ft. diameter outlet pipe for the spillway. During the on site inspection it was revealed that the original downstream outlet for the spillway was extended by a 3 ft. diameter pipe during modification to the downstream face of the Dam for a utility line installation in 1963.

PHOTO NO. 13 - Dumped rubble on downstream slope, and 12 inch diameter discharge pipe from catch basin in upper left hand corner.

PHOTO NO. 14 - Downstream slope as viewed from a point about 100 ft. left of right abutment.

PHOTO NO. 15 - Area of standing water downstream of toe.

PHOTO NO. 16 - Spillway Discharge Channel as viewed from crest.

PHOTO NO. 17 - This photograph shows the gate house at Trapello Road. The gate house and its controls, built about 1896, are no longer used. The three 36 inch cast iron culverts, dating to the 1930's, allow the flow of water from the small upper section of the reservoir to the main section to the right of the roadway. This roadway and the Route 2 roadway, to the north of Trapello Road will not significantly affect the Test Flood Analysis due to the magnitude of the storm being considered.

APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS

JOB NO. 79.206.1
DATE 1/13/79
BY FDD
CH'D BY

HH&B HAYDEN, HARDING & BUCHANAN, INC.
CONSULTING ENGINEERS
BOSTON — WEST HARTFORD

SHEET NO. 2-2
JOB Dams
SUBJECT Cambridge
CLIENT Corps

CAMBRIDGE RESERVOIR

Built: 1895 to 1897
Water Supply (yield 4.0 mgd)
Surface Area: 554± ac.
Drainage Area: 6.82 s.m. (4365ac)
Fed by: Hobbs Brook

Dam Height: 30 ft. } Size Class: Intermediate
Dam Storage: 10,600 ac-ft }

Hazard Potential: High

Test Flood: P.M.F. For Terrain use "Rolling"

PMF Inflow = 6.82 × 1750 = 11,935 cfs

Outflow = 2400 cfs Elev = 186.5 ±

Outflow Chamber and Waste Spillway can pass 1120 cfs or 47 % of Test Flood Outflow

Dam Failure Outflow

$$Q_b = \frac{8}{27}(0.4 \times 400')\sqrt{32.2}(30')^{1.5} = 44,203 \text{ cfs}$$

Damage Due to Failure Outflow

		Flood Stage	
2	Large Industrial Buildings	5' to 10' ±	sta 8+00 downstream
1	House	10' ±	sta 50+00 "
2	Houses	2'-5' ±	sta 66+00 to 77+00
5	Houses & Northern Ave	5'-10' ±	sta [illegible]
6	Houses, Cummings - portions of other roads	5'-10' ±	sta 77+00 to [illegible]
3	Houses	10'-20' ±	sta 77+00 to [illegible]

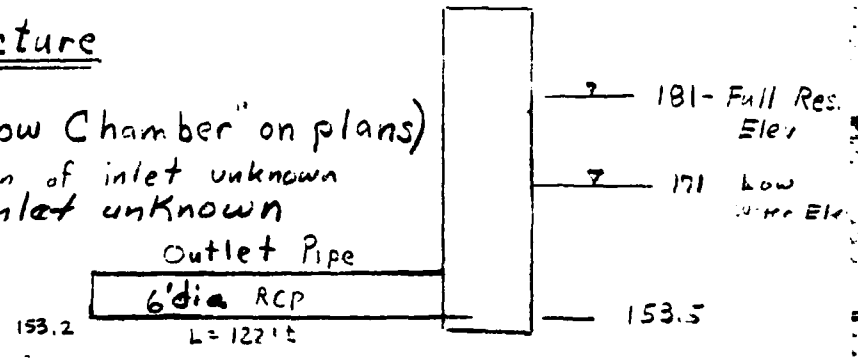
Stage - Storage

Elev.	Area ac.	Ave. Area ac.	Depth ft	Storage ac-ft	Accum. Stor. ac-ft.
160	109.27				
		331.50	12	3978	3,978
172	553.72				
		733.24	8	5866	9,344
180	912.76				
		1092.74	10	10,927	20,771
190	1272.72				

Dam Out Flow

Inlet Structure

(called "Overflow Chamber" on plans)
- actual operation of inlet unknown
- details of inlet unknown



Orifice Flow

$Q = CA\sqrt{2gh}$

A = 28.26 sf

use C ~ 0.6 CA = 22.96

Assume inlets fully opened & discharge controlled by outlet pipe

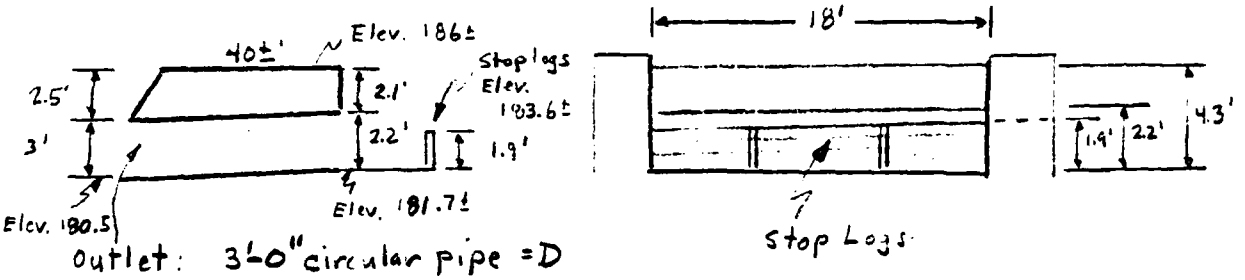
h ft	Q cfs	Elev	h ft	Q cfs	Elev.
10	583	163.5	34.5	1082	199
17.5	771	171	29.5	1001	193
27.5	966	181	33.5	1018	194
32.5	1050	186	31.5	1034	195
33.5	1066	187			

JOB NO. 79.206.1
DATE 11/13/79
BY FOD
CH'D BY

HH&B HAYDEN, HARDING & BUCHANAN, INC.
CONSULTING ENGINEERS
BOSTON — WEST HARTFORD

SHEET NO. D3
JOB Dams
SUBJECT Cambridge Res
CLIENT Corps

Auxiliary Spillway
(modified from original structure)



Outlet: 3'-0" circular pipe = D

Assume No Stoplogs Discharge Controlled by Outlet Pipe which has 3'-0" dia

Ignoring transitions, losses, etc., determine discharge for culvert with inlet control - use Chart #2, USBPR HEC-5

Elev.	HW	HW/D	Q cfs
183	1.5	0.50	11
183.6	2.1	0.70	20
184	2.5	0.83	27
185	3.5	1.17	45
186	4.5	1.50	55
187	5.5	1.83	65
188	6.5	2.17	78

Check: Discharge with stoplogs to Elev. 183.6 have weir flow: $Q = CLH^{3/2}$

H ft	$H^{3/2}$	C	L ft	Q cfs	Elev.
0.4	.25	3.75	18	17	184
1.4		3.96	"	119	185
2.4		4.15	"	250	186

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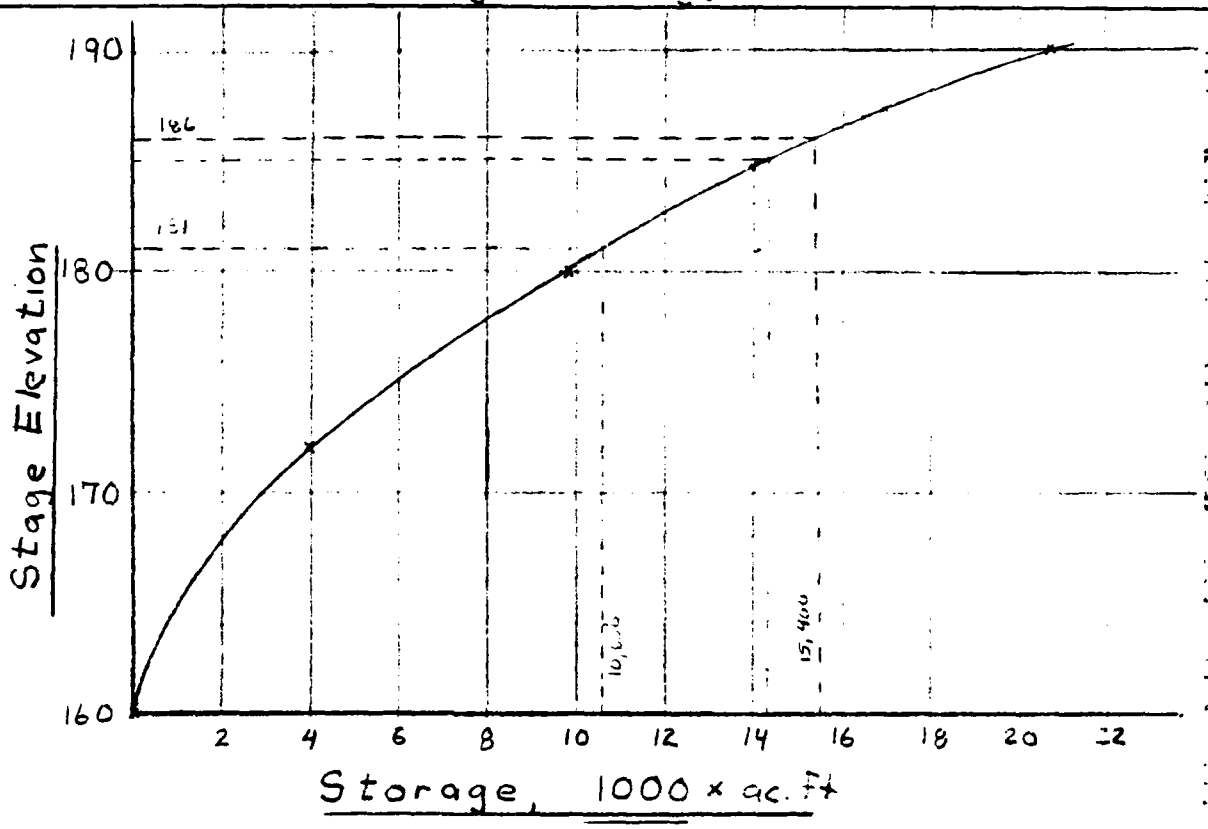
SHEET NO 24
JOB Dams
SUBJECT Cambridge Res
CLIENT Corps

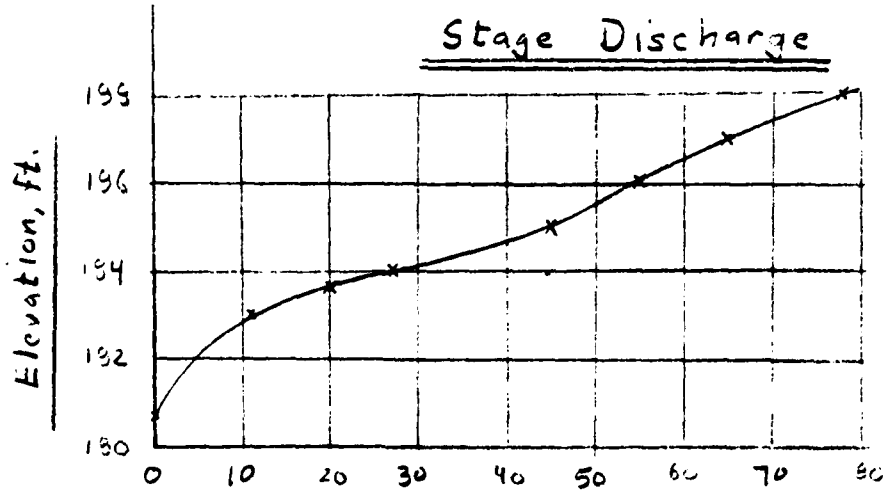
Stage Storage



at elev 186 (top of dam), storage = 15,400 ac-ft

at elev 181 (full pool), storage = 10,600 ac-ft

Stage Discharge



(Control by 36" ϕ pipe or overflow weir)

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SHEET NO. D5
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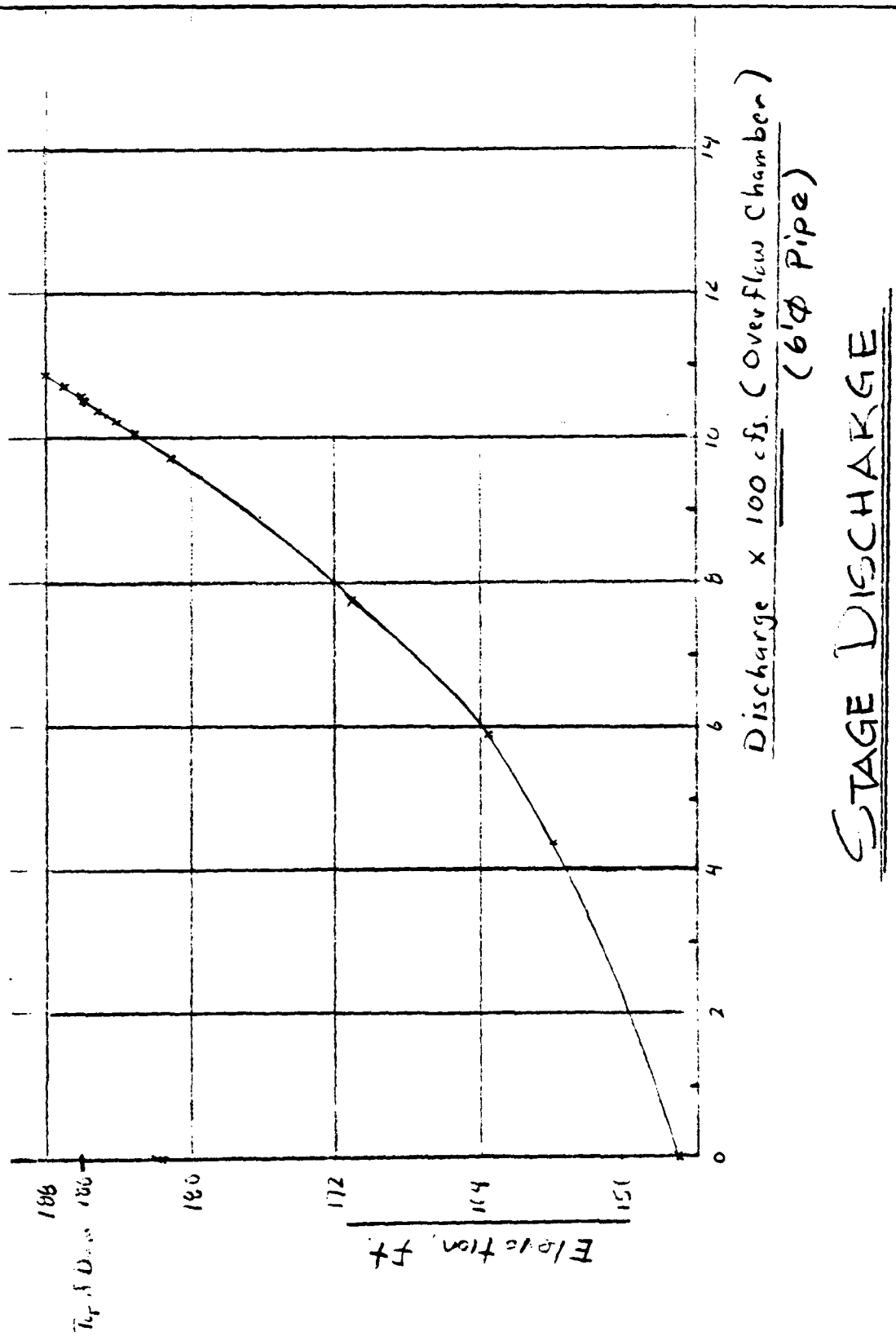


36"ϕ pipe + spillway

@ Elev. 194± : capacity of pipe ≅ flow over weir.

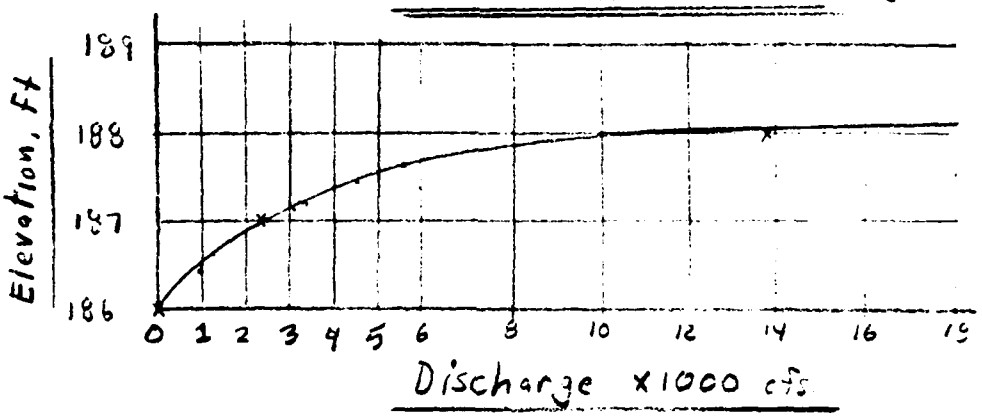
Above this elevation, discharge over weir significantly greater than pipe capacity. Thus outflow controlled by pipe. No stoplogs assumption OK.

Above elevation 186 - get Flow over roadway -

cfs ds Weir: $Q = CLH^{3/2}$ top of road 186±

Elev	H ft	$H^{3/2}$	C	L ft	Q cfs
187	1.0	1.0	2.63	900±	2367
188	2.0	2.83	"	1850±	13,769

STAGE DISCHARGE - Weir Flow over roadway



Out Flow

Q_{PI} = 11,935 cfs (PMF Inflow)

Assume Reservoir Elevation = 194± (Full Pool)

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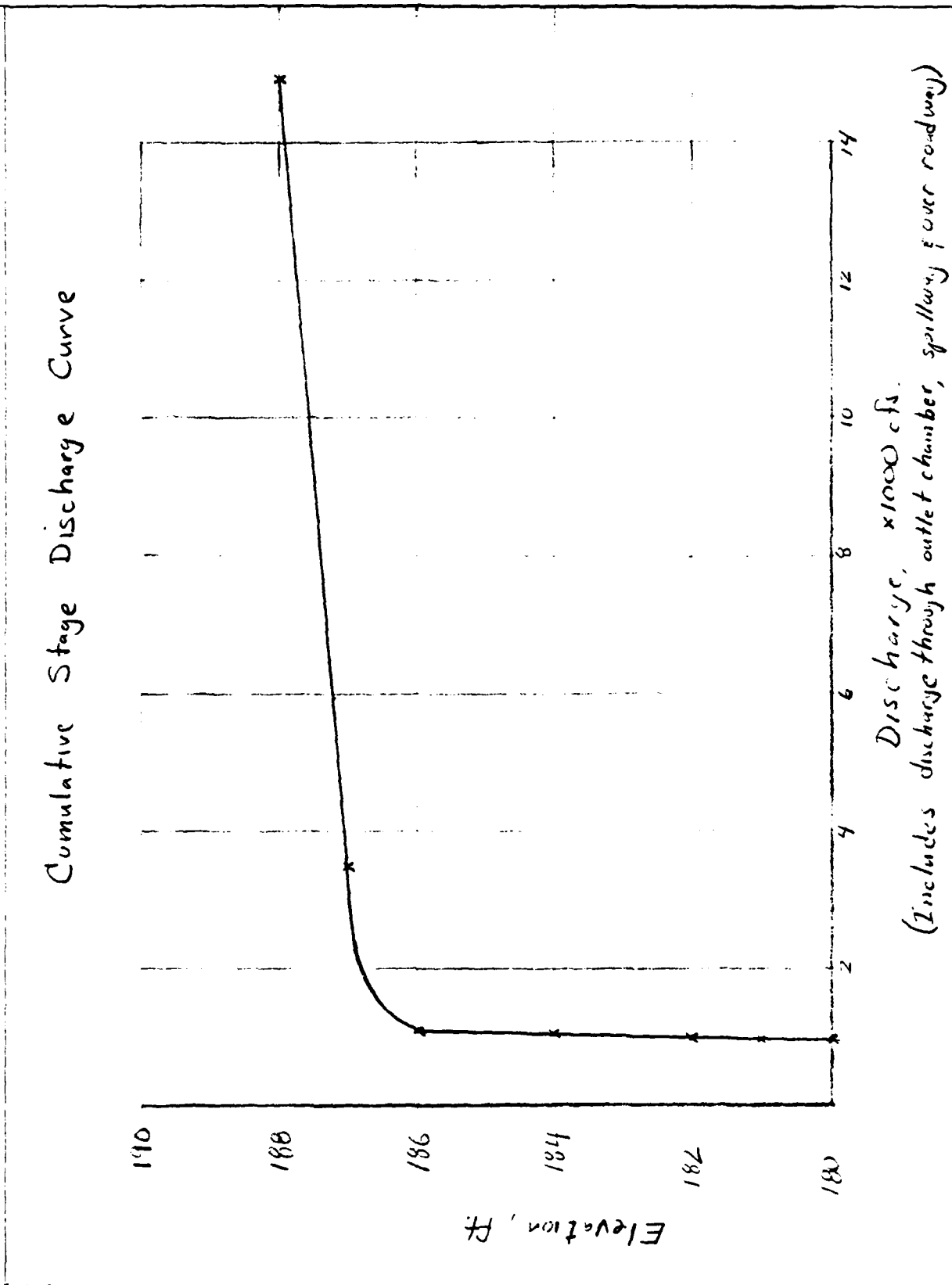
SHEET NO. 26a
JOB Dams
SUBJECT Cumberland Rd.
CLIENT Corps



Exact Operation of Overflow Chamber Outlet is not Known.

① Assume inlets fully open - outflow controlled by capacity of 7' pipe.

Q_{P_1} = 11,935 cfs. Initial Elev. = 191.0
Initial Storage = 10,600 ac-ft; $Stor_0 = 0$

$Elev_1$ = 188.0 ± ($Q_{P_1}' = Q_{P_1} - Q_{outlet}$ = 10,770 cfs)

$Stor_1$ = 17,800 ac-ft − 10,600 ac-ft = 7,200 ac-ft

$= \frac{7200}{4345} \times 12 = 19.9'' > 19''$ from PMF inflow.

take $Stor_{ave} = \frac{Stor_0 + Stor_1}{2} = \frac{0 + 7,200}{2} = 3600$ ac-ft

$Stor_{ave} = \frac{3600}{4345} \times 12 = 9.94$ in

$Q_{P_2} = 11,935\left(1 - \frac{9.94}{19.0}\right) = 5691$ cfs.

Q_{P_2} = 5,700 cfs ± $Elev_2$ = 197.5 ±

$Stor_2 = 17,000 - 10,600 = 6,400$ ac-ft

$Stor_{ave} = \frac{3600 + 6400}{2} = 5000$ ac-ft $\times \frac{12}{4345} = 13.8''$

$Q_{P_3} = 11,935\left(1 - \frac{13.8}{19}\right) = 3266$ cfs $Elev_3$ = 197.1 ±

$Stor_3 = 16,600 - 10,000 = 6,600$ ac-ft

$Stor_{ave} = \frac{6400 + 6000}{2} = 6200$ ac-ft $\times \frac{12}{4345} = 17.1''$

$Q_{P_4} = 11,935\left(1 - \frac{17.1}{19}\right) = 1194$ cfs. ✓

$Q_{outlets}$ @ elev 186.0+ = 1112 cfs. ✓

$Elev_4 = 186.0 \pm$ $\quad Stor_4 = 15,400 - 10,600 = 4800$

$Stor_{ave} = \frac{6200 + 4800}{2} = 5500$ ac-ft $\times \frac{12"}{43,560}$ = 15.2"

$Q_{P_5} = 11,935\left(1 - \frac{15.2}{19}\right) = 2387$ cfs.

$Elev_5 = 186.5 \pm$ ($Q_{outlets} = 1062 + 63 = 1120$; $Q_{weir} = 1000$) = 2120±

$Stor_5 = 16,000 - 10,600 = 5400$ ac-ft

$Stor_{ave} = \frac{5500 + 5400}{2} = 5450$ ac-ft $\times \frac{12"}{43,560}$ = 15.1" ✓

$Q_{P_6} = 11,935\left(1 - \frac{15.1}{19}\right) = 2450$ cfs. ✓

$Elev_6 = 196.6 \pm$ $Q_{outlets} = 1130$ cfs.
$Q_{weir} = 1320$ cfs)

$Stor_6 = 16,200 - 10,600 = 5600$

$Stor_{ave} = \frac{5400 + 5600}{2} = 5500$ ac-ft $\times \frac{12"}{43,560}$ = 15.2"

$Q_{P_7} = 11,935\left(1 - \frac{15.2}{19}\right) = 2387$ cfs. ✓ $\quad Elev. 186.5 \pm$ ✓

$Stor_7 = 5400$ ac-ft

$Q_{out} = 2400.$ cfs ± @ Elev. 186.5 ± ✓

Capacity of outlets = 1120. cfs ±
Flow over roadway = 1280 cfs ± with depth = 0.5'
over roadway.

JOB NO. 79206.1
DATE 11-13-79
BY WA L
CH'D BY FDM

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SHEET NO 7B
JOB Dams
SUBJECT Cranberry Sp
CLIENT CoE

1/2 PMF

Inflow = 6,000.± cfs

Elev 181 normal level = base level
Elev 186 roadway Available Str = 4800 a-f

1/2 PMF runoff = 9.5" × $\frac{1}{12}$ × $\frac{F/11}{4367}$ a = 3460 ± a-f

Elev of inflow = 185 ± if all outlets are blocked or w/ very small outflow - can't calculate accurately due to lack of data for low flow discharge — reservoir could store all of 1/2 PMF inflow w/o overtopping.

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DATE 11/15/79
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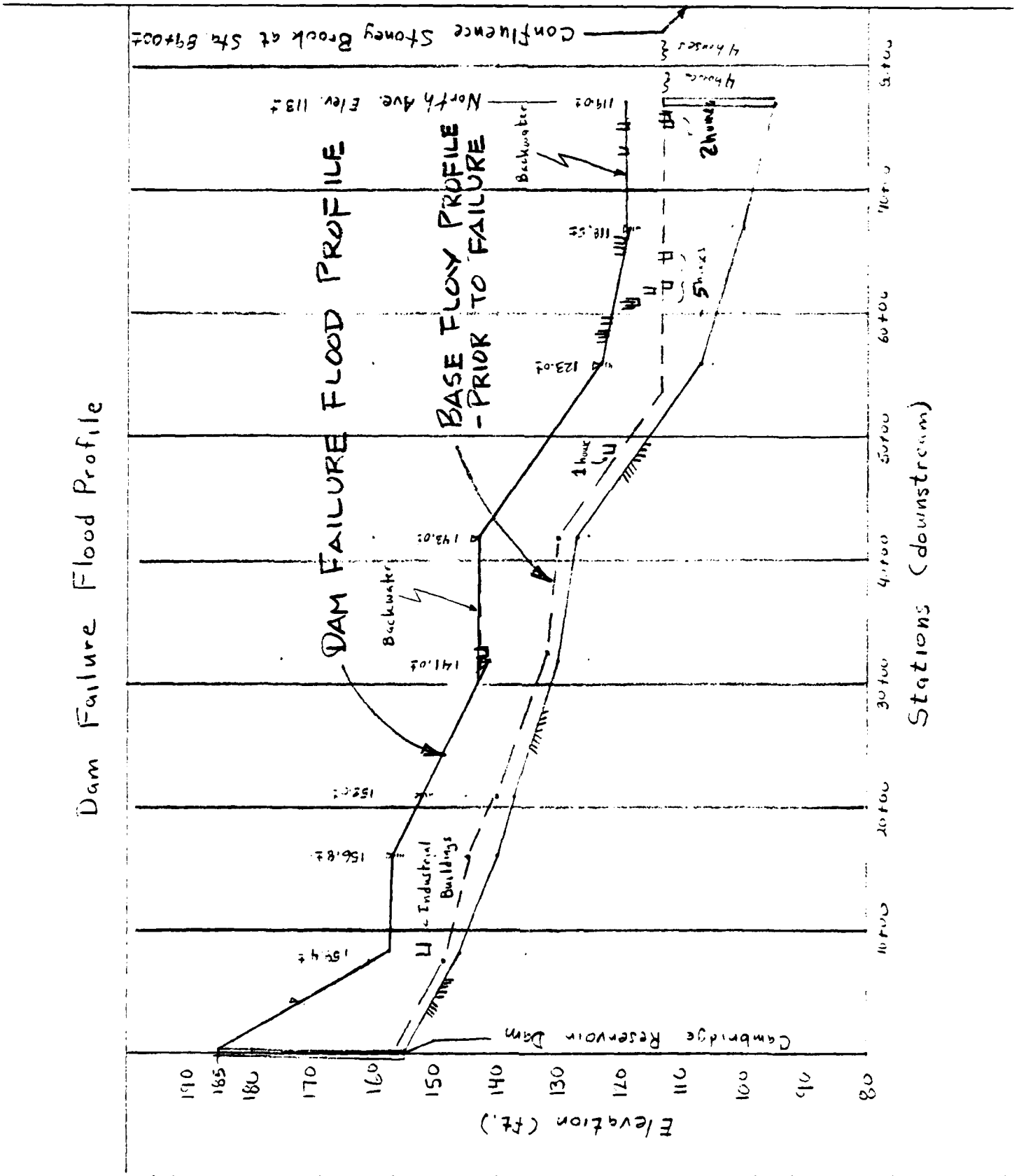
SHEET NO. D8
JOB Dams
SUBJECT Cambridge Res.
CLIENT Corps

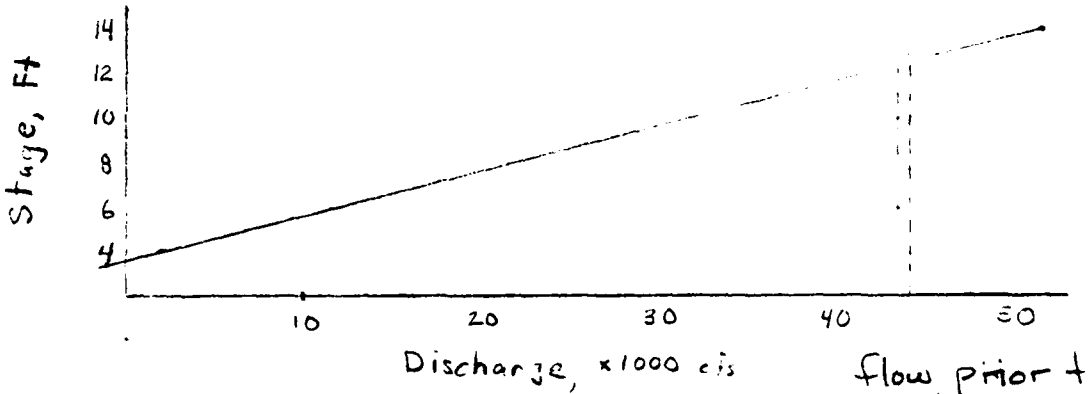


Sta 8+00

$n = .10 \quad S^{1/2} = \left(\frac{4}{800}\right)^{1/2} = 0.0707 \quad V = \frac{1.486}{0.10}(R^{2/3})(.0707) =$

$V = 1.051\ R^{2/3}$

D	WP (Ft.)	A (sf)	$R^{2/3}$	F'	V (Fps.)	Q (cfs)
4'	600	1200	1.587	1.051	1.668	2002
14'	1,750	12,950	3.797	"	3.991	51685
24'	2,000	31,700	6.310	"	6.632	210,224



flow prior to failure ≈ 1100 cfs
base storage = 14350 ± a-F,

Q_{P_1} = 44,203 cfs ✓ at failure

D = 12.5' A = 10,125 sf

$Vol_1 = \frac{4800 + 10,125}{2} \times 800 \times \frac{1}{43,560} = 137.1$ ac-ft ✓ $< \frac{1}{2}$ (14,350) $> <$

$Q_{P_2} = 44,203\left(1 - \frac{137.1}{14350}\right) = 43731$ cfs ✓ $D_2 = 12.4'$

$Vol_2 = \frac{4800 + 9936}{2} \times 800 \times \frac{1}{43,560} = 135.3$ ac-ft ✓

$Vol_{ave} = (137.1 + 135.3) \times 1/2 = 136.2$ ac-ft ✓

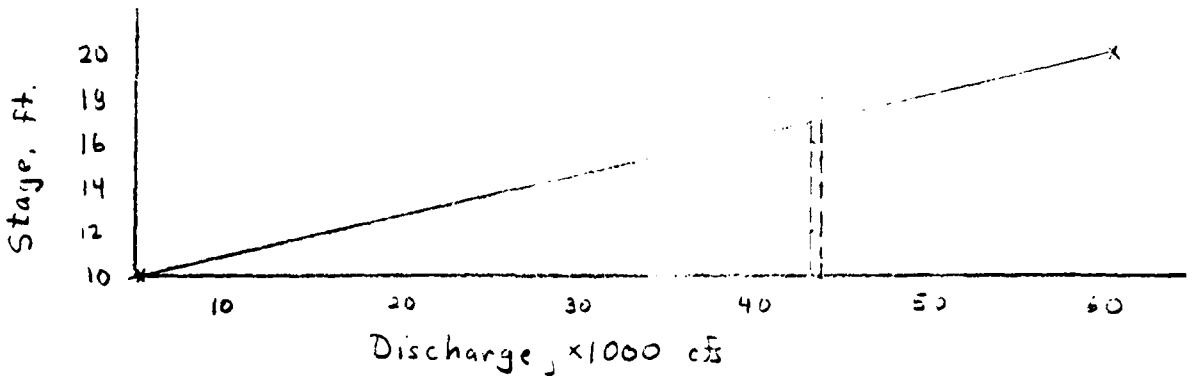
$Q_{P_3} = 44,203\left(1 - \frac{136.2}{14350}\right) = 43783$ cfs ✓ Elev. = 153.4 ±

Sta 16+00

$n = 0.08 \quad S^{1/2} = \left(\frac{6}{300}\right)^{1/2} = 0.087 \quad F' = \frac{1.486}{0.08}(.087) = 1.62$

$V = F' R^{2/3}$

D Ft	W.P ft	A sf	$R^{2/3}$	F'	V fps	Q cfs
10	300	1500	2.92	1.62	4.73	7096
20	1400	10,000	3.71	"	6.01	60,055
30	1525	24,625	6.39	"	10.35	254,357



$Q_{P_1} = 43783$ cfs. $\quad D = 17.0 \quad A = 1330$

$Vol_1 = \frac{10030 + 6330}{2} \times 800 \times \frac{1}{43560} = 150.2$ ac-ft $< \frac{1}{2} \times 14350$ OK

$Q_{P_2} = 43,783 \left(1 - \frac{150.2}{14350}\right) = 43,325$ cfs $\quad D_2 = 16.8'$

$Vol_2 = \frac{10030 + 6090}{2} \times 800 \times \frac{1}{43,560} = 149.0$

$Vol_{ave} = \frac{148.0 + 150.2}{2} = 149.1$

$Q_{P3} = 43,783 \left(1 - \frac{149.1}{14350}\right) = 43,323$ cfs.

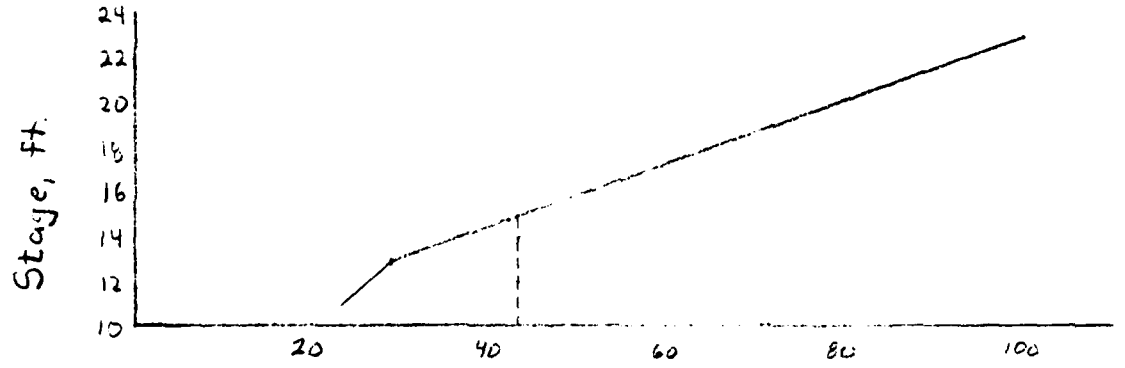
Elev. = 156.9' ±

Sta. 22+00

$n = 0.10 \quad S^{1/2} = \left(\frac{3}{600}\right)^{1/2} = (0.005)^{1/2} \quad F' = \frac{1.49}{0.10}(.005)^{1/2} = 1.05$

$V = F' R^{2/3}$

D ft	WP ft	A sf	$R^{2/3}$	F'	V fps	Q cfs
3	400	600	1.31	1.05	1.37	825 ✓
13	800	6,600	4.08	"	4.28	28,274
23	1290	17,050	5.59	"	5.87	100,075
33	1380	30,400	7.86	"	8.25	250,891



$Q_p = 43,328$ cfs $D = 15'$ $A = 8690$ sf

$V_{01} = \frac{6210 + 8690}{2} \times 600 \times \frac{1}{43560} = 102.6$ cfs $< 14350 \times \frac{1}{2}$

$Q_{p_2} = 43,328\left(1 - \frac{102.6}{14350}\right) = 43018$ cfs $D_2 = 15'$

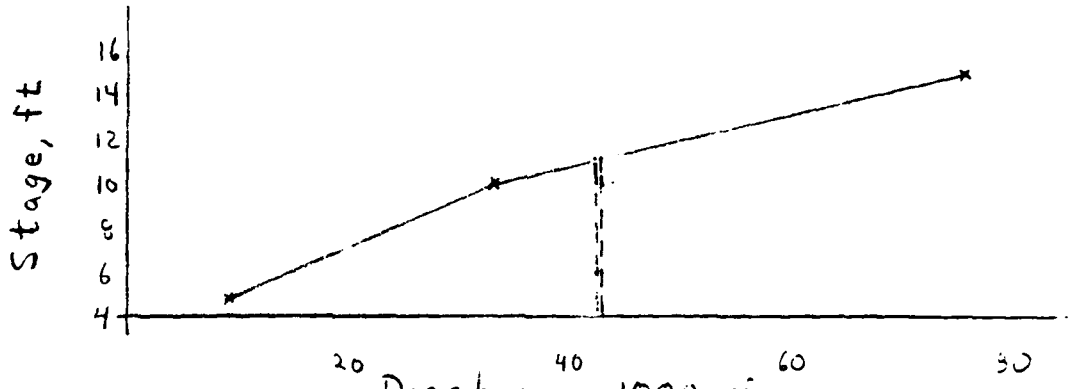
$Q_{p_2} = 43,019$ cfs Elev = 152' ✓

Sta 32+00

$n = 0.08$ $S^{1/2} = \left(\frac{7}{1000}\right)^{1/2} = 0.084$ $F' = \frac{1.486}{.08}(.084) = 1.56$

$V = F' R^{2/3}$

D	WP	A	$R^{2/3}$	F'	V	Q
ft	ft	sf			f/s	cfs
10	800	5750	3.72	1.56	5.81	33,409
15	970	10,175	4.79	"	7.48	76,063
5	580	2325	2.52	"	3.94	9,161



$Q_{P_1} = 43,018$ cfs $D_1 = 11' \pm$ $A_1 = 6635$ sf

$Vol_1 = \frac{8690 + 6635}{2} \times 1000 \times \frac{1}{43,560} = 175.9$ ac-ft $< \frac{1}{2}$ 14350 ac-ft

$Q_{P_2} = 43,018\left(1 - \frac{175.9}{14350}\right) = 42,491$ cfs $D_2 = 11'$

$Q_{P_2} = 42,491$ cfs Elev = 141

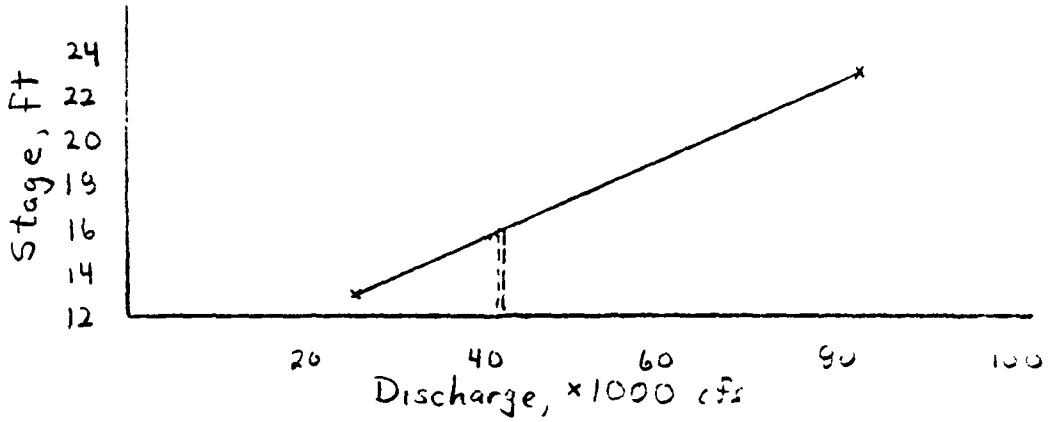
Sta 28+00 [illegible]

Stg 42+00

$n = 0.10 \quad S^{1/2} = \left(\frac{3}{1000}\right)^{1/2} = 0.055\% \quad F' = \frac{1.486}{0.10}(.055) = .9$

$V = F' R^{2/3}$

D (Ft)	WP (Ft)	A (sf)	$R^{2/3}$	F'	V (fps)	Q (cfs)
3	500	750	1.31	0.82	1.07	806
13	800	7250	4.35	"	3.56	25,841
23	1600	19250	5.25		4.31	82,891



$Q_{P_1} = 42,491$ cfs $\quad D_1 = 16' \quad A_1 = 10610$

$Vol_1 = \frac{6635 + 10850}{2} \times 1000 \times \frac{1}{43,560} = 200.7$ ac-ft $< \frac{1}{2} \times 14350$ OK

$Q_{P_2} = 42,491\left(1 - \frac{200.7}{14350}\right) = 41,897$ cfs $\quad D_2 = 15'8.6'$

$Q_{P_2} = 41,897$ cfs $\quad$ Elev = 143' > 141 upstream

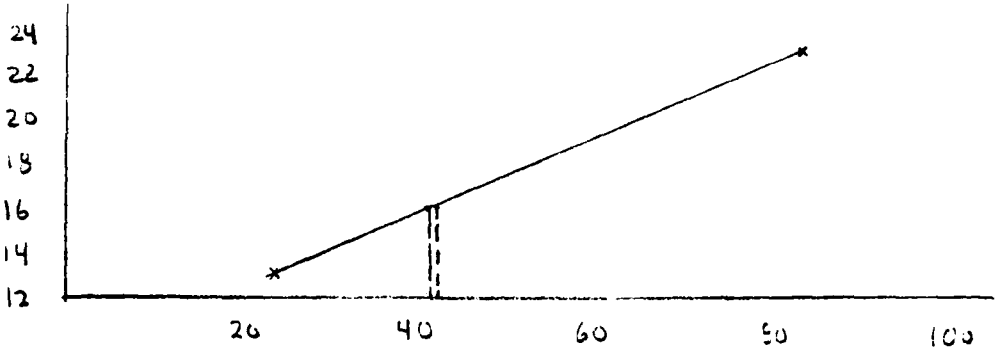
— results in Backwater Effect upstream

Sta. 56+00

$n = 0.08 \quad S^{1/2} = \left(\frac{20}{1400}\right)^{1/2} = .120 \quad F' = \frac{1.486}{.08}(.120) = 2.23$

$V = F' R^{2/3}$

D ft	WP ft	A sf	$R^{2/3}$	F'	V fps	Q cfs
3	150	225	1.31	2.23	2.92	657
13	320	2575	4.02	"	9.96	23,060
23	520	6775	5.54	"	12.35	83,654



$Q_{P_1} = 41,897 \quad D_1 = 16' \pm \quad A_1 = 3835$ sf

$Vol_1 = \frac{10850 + 3835}{2} \times 1400 \times \frac{1}{43,560} = 236.0$ ac-ft OK

$Q_{P_2} = 41,997\left(1 - \frac{236}{14350}\right) = 41,208$ cfs $\quad D_2 = 16'$

$Q_{P_2} = 41,208$ cfs $\quad$ Elev $= 123'$

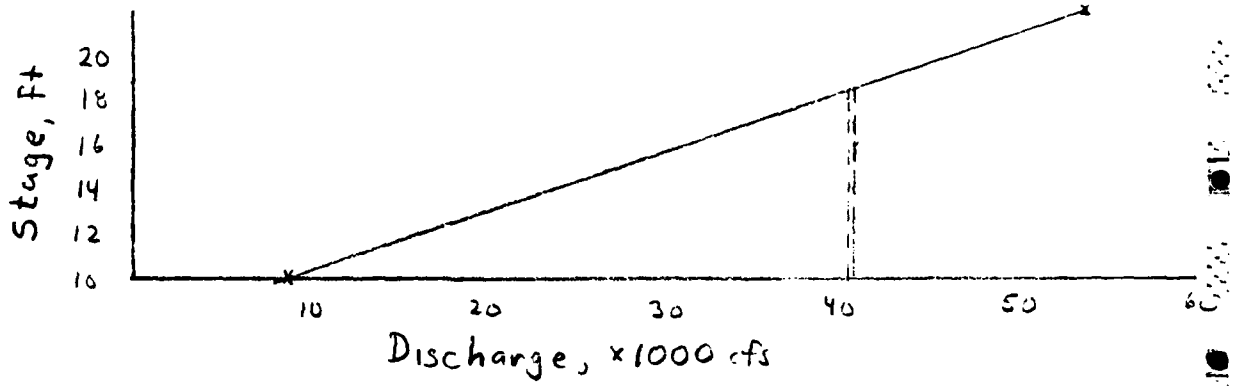
Sta 53+00 1 house flooded by 5' ±

Sta 67+00

$n = 0.08$ $S^{1/2} = \left(\frac{7}{1100}\right)^{1/2} = .080\%$ ✓ $F' = \frac{1.486}{.08} \times (.08) = 1.49$ ✓

$V = F' R^{2/3}$

D	WP	A	$R^{2/3}$	F'	V	Q
ft	ft	sf			fps	cfs
10	400	2000	2.92	1.49	4.36	9,714
20	580	6900	5.21	"	7.76	53,578 ✓



$Q_{P_1} = 41,208$ cfs. $D_1 = 18.5'$ $A_1 = 6165$ sf

$Vol_1 = \frac{3935 + 6165}{2} \times 1100 \times \frac{1}{43,560} = 126.3$ ac-ft ✓

$Q_{P_2} = 41,208 \left(1 - \frac{126.3}{1435.0}\right) = 40,845$ cfs. ✓

$D_2 = 19.4 \sim 19.5'$

$Q_{P_2} = 40,845$ cfs Elev = 119.5'

In Reach 56+00 to 67+00

4 houses flooded @ 5' ±
3 houses on fringe of flooding
300' ± of road inundated

Sta. 77+00 - North Ave.

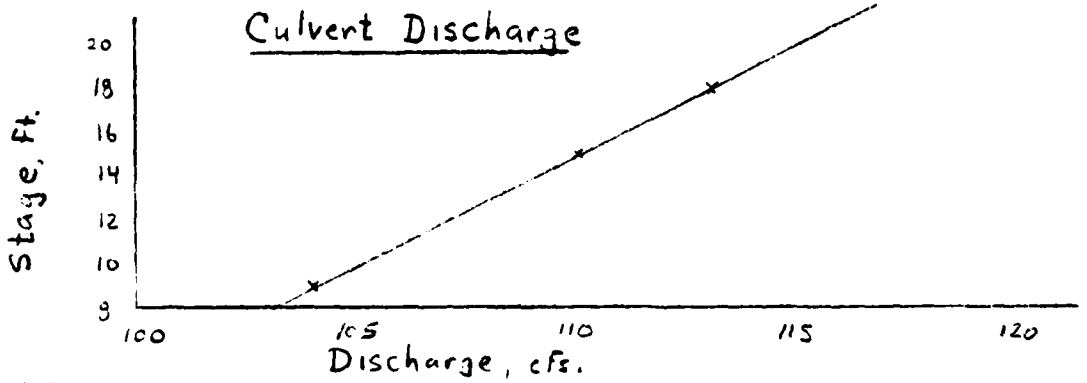
$n = 0.08 \quad S^{1/2} = \left(\frac{5}{1000}\right)^{1/2} = .0707$

have 7'x9' box culvert - use $Q = CA\sqrt{2gH}$
with $C = 0.6 \quad A = 63$

H (ft)	$\sqrt{2gH}$	CA	Q (cfs)	Elev.	
9	24.1	37.8	910	104	
15	31.1	"	1175	110	
18	34.0	"	1287	113	- overtops road
20	35.9	"	1357	115	
23	38.5	"	1455	118	
25	40.12	"	1517	120	

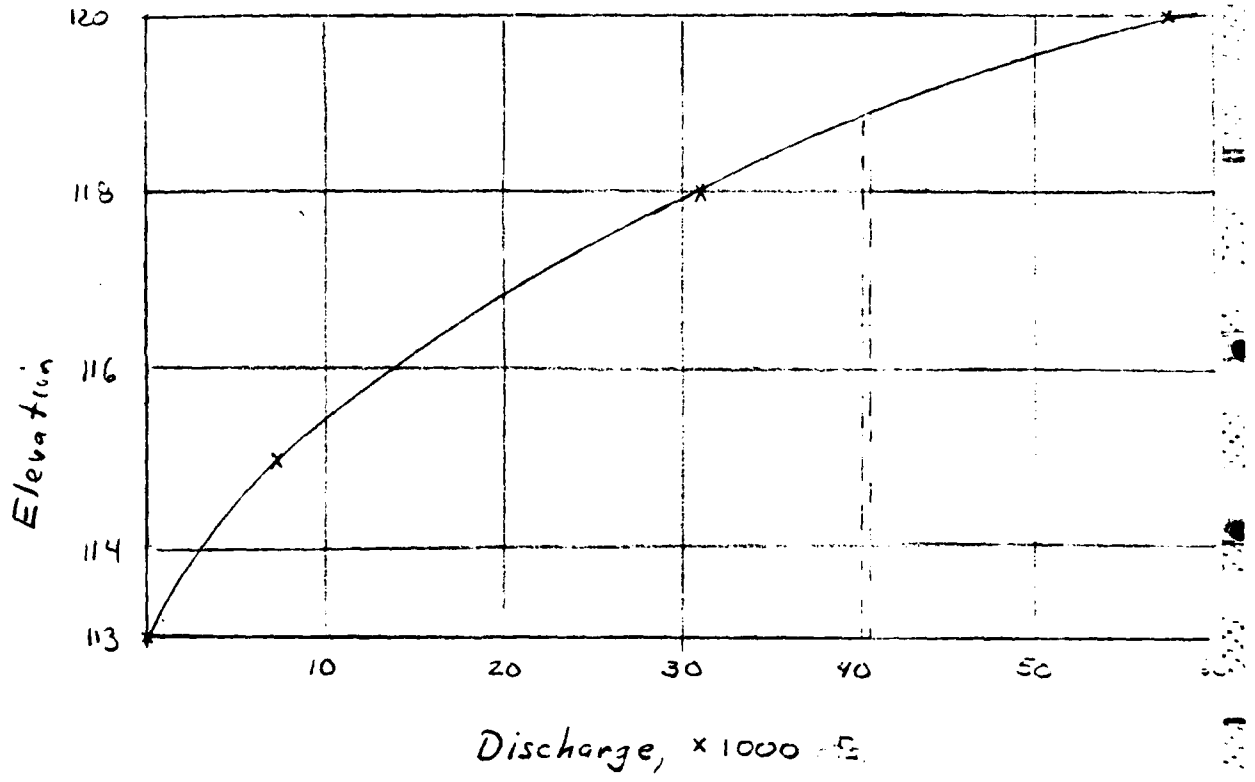
For flow over roadway $Q = CLH^{3/2}$

Elev.	H (ft)	$H^{3/2}$	L (ft)	C	Q (cfs)	cum Q (cfs)
115	2	2.82	900	2.63	5937	7294
118	5	11.19	1020	2.63	29,991	31,446
120	7	18.52	1150	2.63	56,014	57,531



Combined Discharge – Flow through culvert insignificant when compared to flow over roadway
For base flow = 1100 cfs, get elev 113.2±

Flow over Roadway (Weir Flow)



Assume area behind road below elevation 113 "dead" storage due to base flow (also assume road embankment doesn't fail)

Q_{p_1} = 40,945 cfs D_1 = 23.7'± A_1 = 3053 – 5[illegible]
A_1 = 707½ sf

$Vol_1 = \frac{6165 - 707\frac{1}{2}}{2} \times 1000 \times \frac{1}{43,560}$ = 152.0 [illegible]

$Q_{p_2} = 40,945\left(1 - \frac{152.0}{1435\text{[illegible]}}\right)$ = 40,412 cfs

Q_{P_2} = 40,412 D_2 = 23.8± A_2 = 12,955 − 5475
A_2 = 6480 sf

Vol_2 = 150.9 ac-ft

$Vol_{ave} = \frac{152.0 + 150.9}{2}$ = 151.5 ac-ft

$Q_{P_3} = 40,845\left(1 - \frac{151.5}{1435}\right)$ = 40,414 cfs ✓ D_3 = 23.8'

Q_{P_3} = 40,414 cfs. Elev = 119'±

- Backwater Effect for upstream section

In reach 67+00 to 77+00 ± downstream to confluence with Stoney Brook:

5 houses on fringe of flooding
5 houses flooded by 2'-5'±
12 houses flooded by 5'-10'±
+ 800' North Ave flooded
+ 500' Church St "
+ 1000' other roads "

JOB NO. 79.206.1
DATE 11/14/79
BY EDD
CH'D BY MA

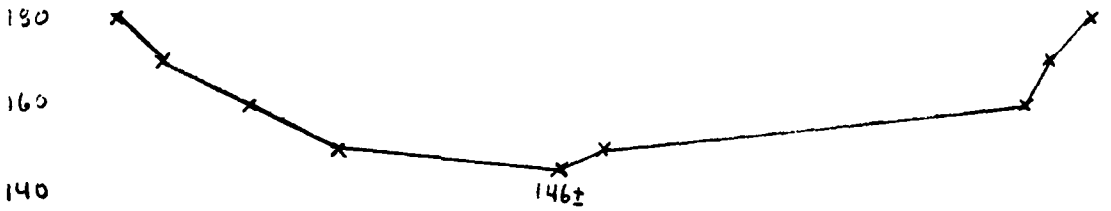
HH&B HAYDEN, HARDING & BUCHANAN, INC.
CONSULTING ENGINEERS
BOSTON — WEST HARTFORD

SHEET NO 19
JOB Dams
SUBJECT Cambridge, Mass.
CLIENT Corps

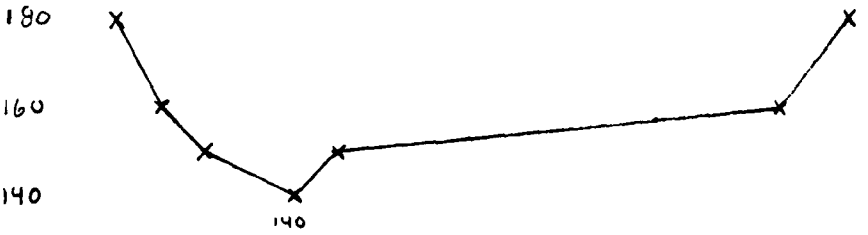
Cross Sections (looking upstream)

Scale: Hor. 1" = 400'
Vert. 1" = 40'

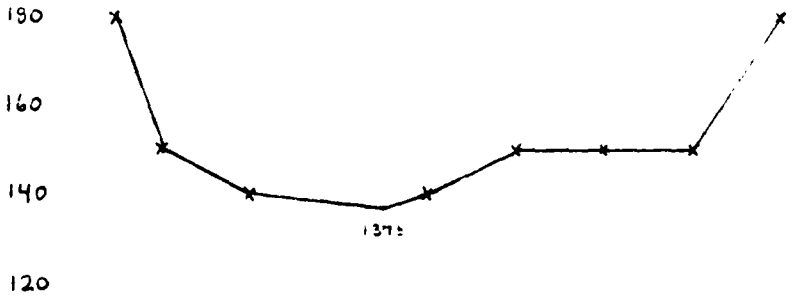
Sta. 8+00



Sta. 16+00



Sta. 22+00



JOB NO. 79.206.1
DATE 11/14/79
BY FDD
CH'D BY MR

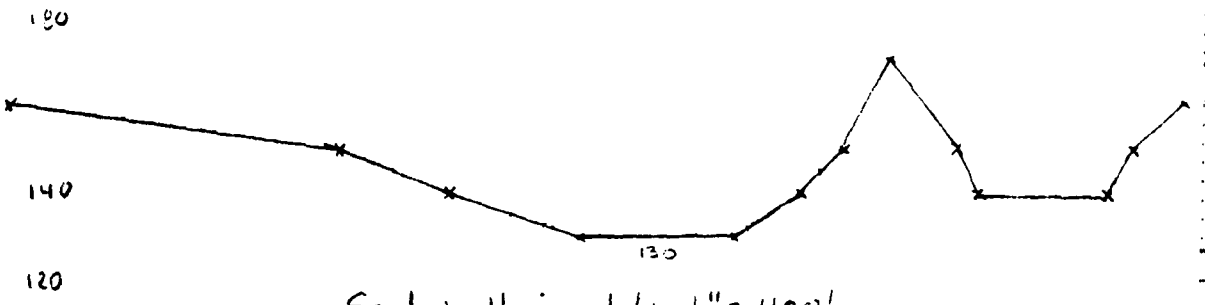
HH&B HAYDEN, HARDING & BUCHANAN, INC.
CONSULTING ENGINEERS
BOSTON — WEST HARTFORD

SHEET NO 220
JOB Dam
SUBJECT Cambridge Res
CLIENT Corps

Cross Sections (looking upstream)

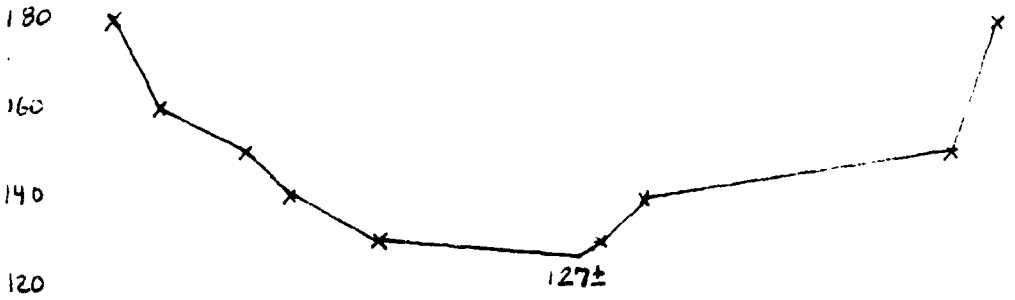
Scale: Vertical 1" = 40'

Sta 32+00



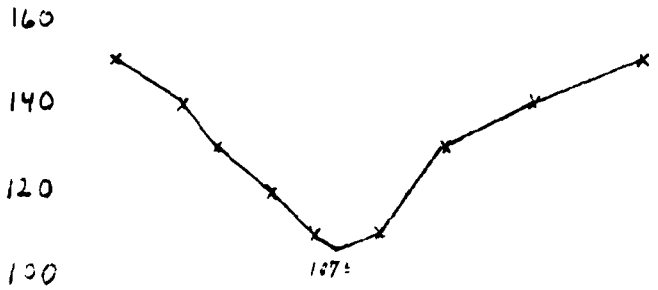
Scale: Horizontal 1" = 400'

Sta 42+00



Scale: Horizontal 1" = 400'

Sta 56+00



Scale: Horizontal 1" = 400'

JOB NO. 71.206.1
DATE 11/14/71
BY FDD
CH'D BY MA

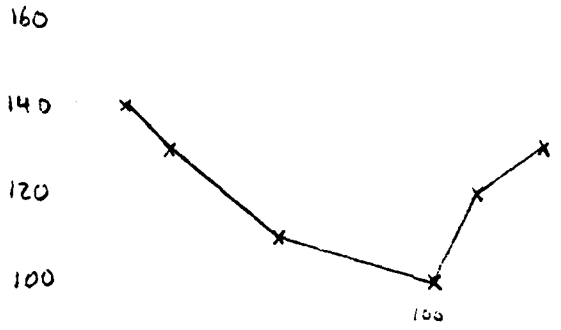
HH&B HAYDEN, HARDING & BUCHANAN, INC.
CONSULTING ENGINEERS
BOSTON — WEST HARTFORD

SHEET NO. 21
JOB Dam
SUBJECT Concrete Fig
CLIENT Corps

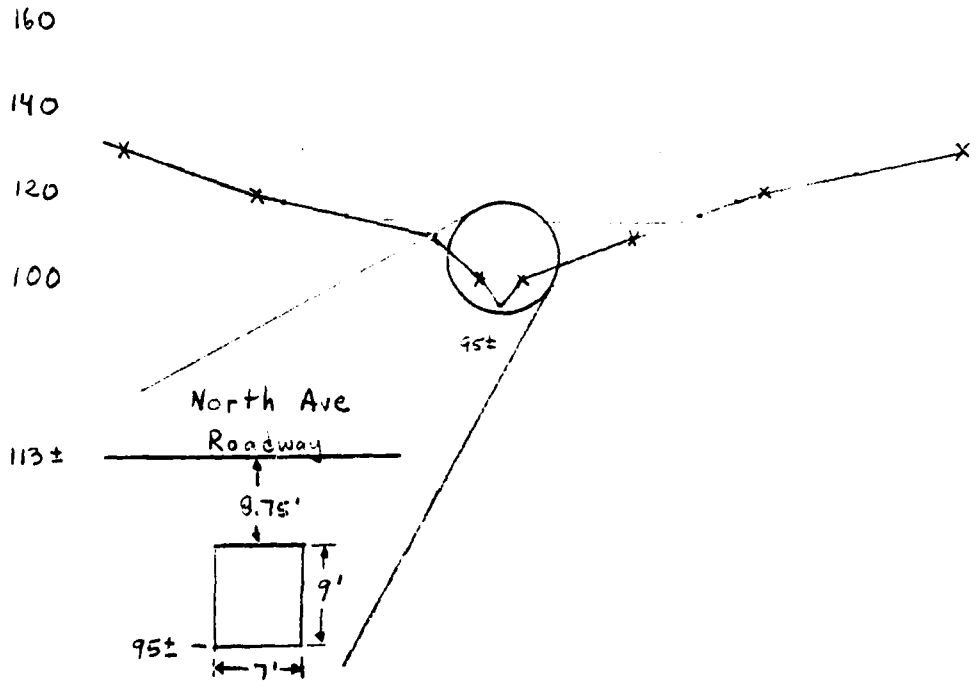
Cross Sections (looking upstream)

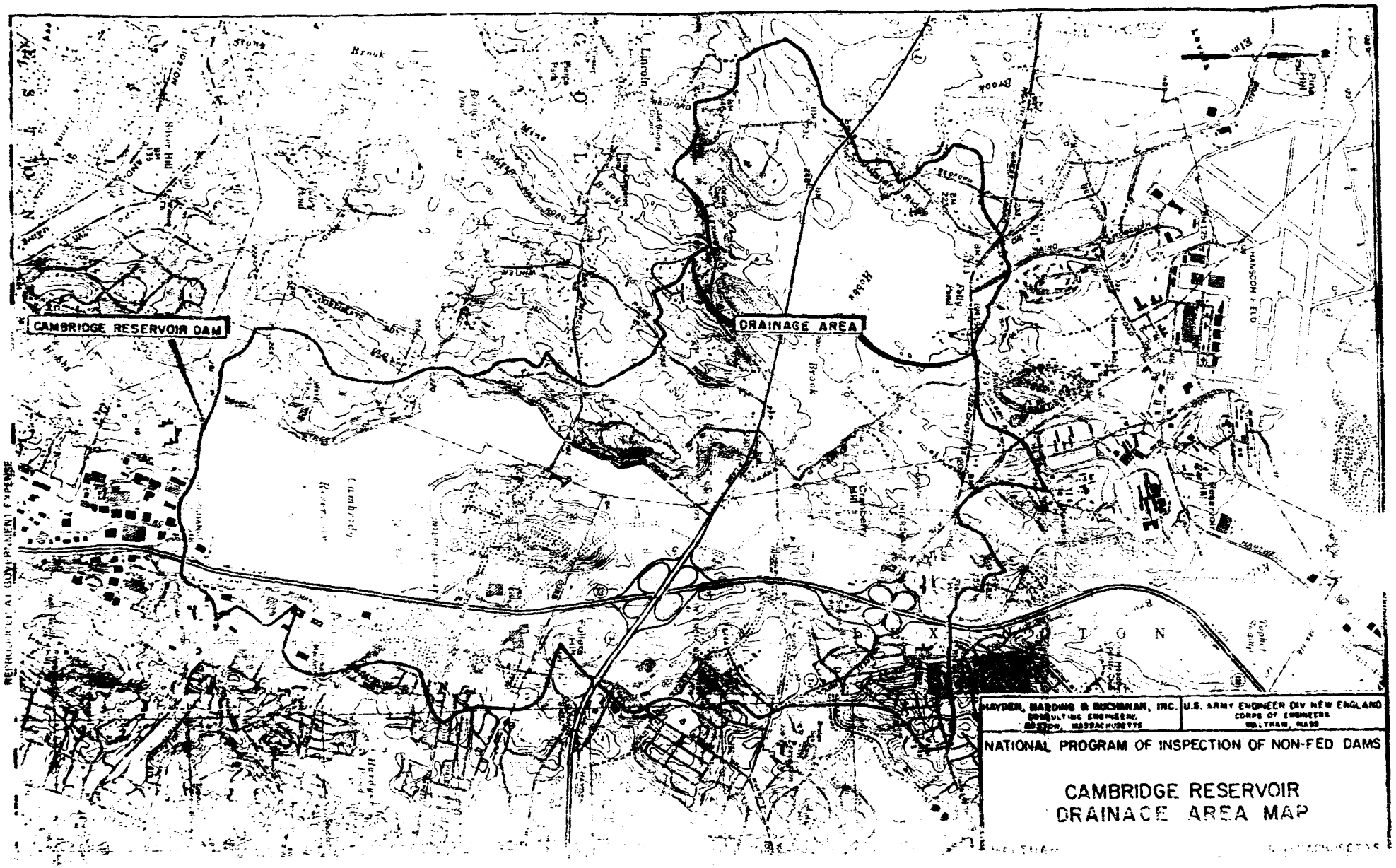
Scale: Hor. : 1" = 400'
Vert. : 1" = 40'

Sta. 67+00

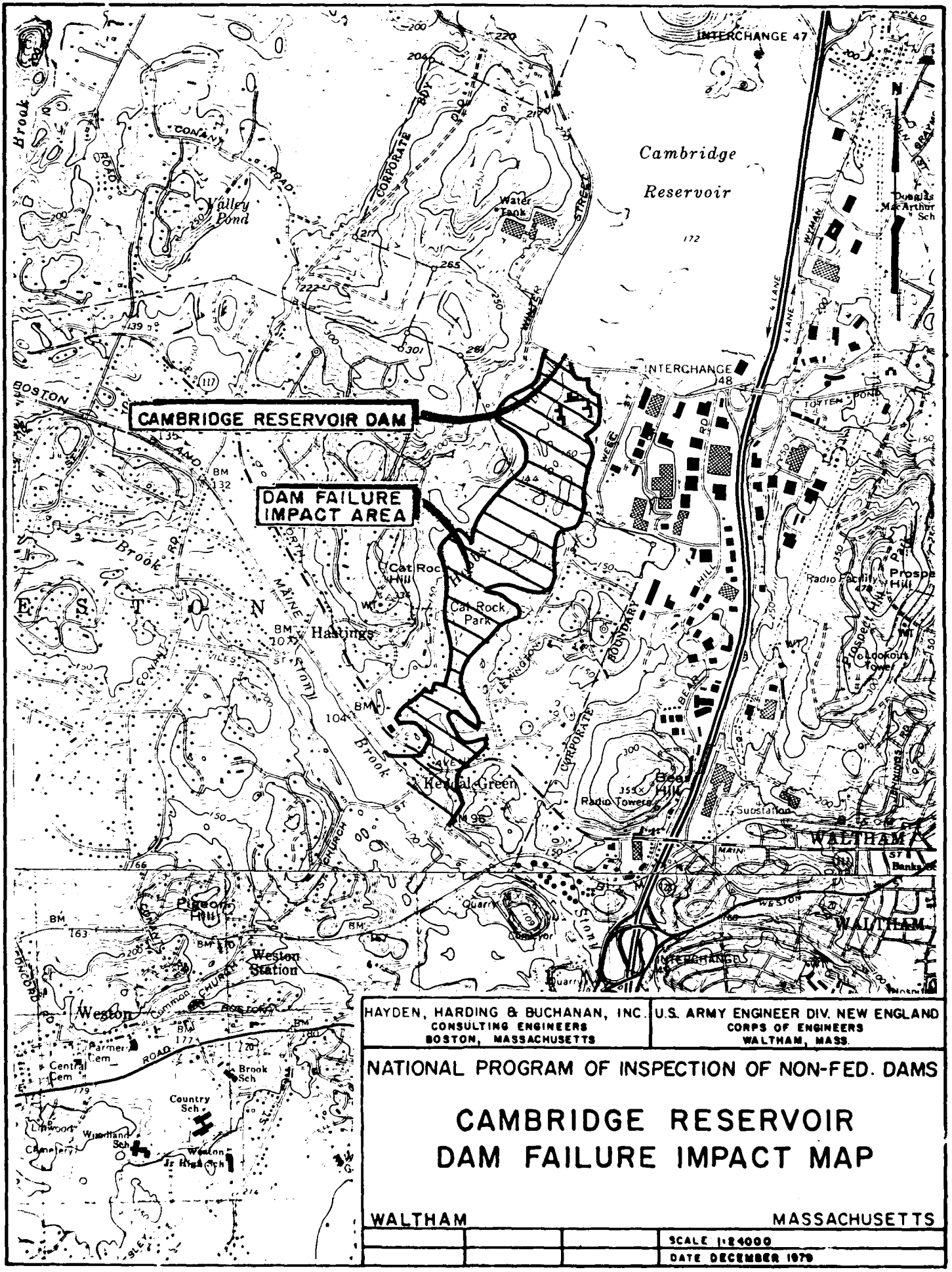


Sta. 77+00 (North Ave.)




CAMBRIDGE RESERVOIR DAM
DRAINAGE AREA
Cambridge
Brook
Stony
Silver Hill
Fuller's Hill
Hanscom Field
REPRODUCTION (COMPLIMENTS) EXPENSE
HAYDEN, HARDING & BUCHANAN, INC.
CONSULTING ENGINEERS
BOSTON, MASSACHUSETTS
U.S. ARMY ENGINEER DIV NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS
NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS
CAMBRIDGE RESERVOIR
DRAINAGE AREA MAP

D-22



HAYDEN, HARDING & BUCHANAN, INC. CONSULTING ENGINEERS BOSTON, MASSACHUSETTS	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

CAMBRIDGE RESERVOIR
DAM FAILURE IMPACT MAP

WALTHAM MASSACHUSETTS

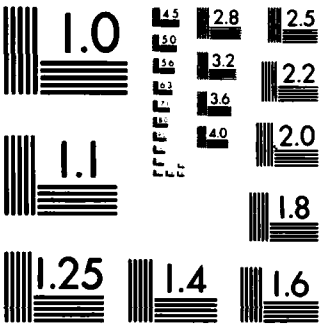
			SCALE 1:24000
			DATE DECEMBER 1979

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS

AD-A155 575 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS 2/2
CAMBRIDGE RESERVOIR D. (U) CORPS OF ENGINEERS WALTHAM
MA NEW ENGLAND DIV JAN 80

UNCLASSIFIED F/G 13/13 NL



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR DIST.	STATE	COUNTY	CONGR DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY	MO	YR
MA	[illegible]	NED	MA	017	04				CAMBRIDGE RESERVOIR DAM	4223.9	7116.4	28	DEC	79

POPULAR NAME	NAME OF IMPOUNDMENT
HOBBS BROOK RESERVOIR	CAMBRIDGE RESERVOIR

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY–TOWN–VILLAGE	DIST FROM DAM (MI.)	POPULATION
01	b	HOBBS BROOK	WALTHAM	0	61600

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	IMPOUNDING CAPACITIES NORMAL (ACRE-FT.)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
REPDOT	1897	S	32	32	15400	10600	NED	N	N	N	N	

REMARKS
21 CONC CONE, ON PLANS

D/S HAS	SPILLWAY CREST LENGTH	SPILLWAY TYPE	SPILLWAY WIDTH (FT.)	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	POWER CAPACITY PROPOSED (MW)	NAVIGATION LOCKS NO.	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
1	1650	U	18	55												

OWNER	ENGINEERING BY	CONSTRUCTION BY
CAMBRIDGE WATER DEPT.		

REGULATORY AGENCY			
DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
NONE	NONE	NONE	NONE

INSPECTION BY	INSPECTION DATE DAY	MO	YR	AUTHORITY FOR INSPECTION
HAYDEN, HARDING + BUCHANAN	30	OCT	79	P.L. 92-367

REMARKS

END

FILMED

8-85

DTIC